Technical Report Documentation Page

1. Report No. FHWA/TX-98/1732-1	2. Government Accession No.	3. Recipient's Catalog No.
4. Title and Subtitle REEVALUATION OF RAMP DESIGN SPEED CRITERIA: REVIEW OF PRACTICE AND DATA COLLECTION PLAN		5. Report Date April 1997
7. Author(s) Michael Hunter and Randy Machemehl		6. Performing Organization Code 8. Performing Organization Report No. 1732-1
9. Performing Organization Name and Address Center for Transportation Research The University of Texas at Austin 3208 Red River, Suite 200 Austin, TX 78705-2650		10. Work Unit No. (TRAIS) 11. Contract or Grant No. 0-1732
12. Sponsoring Agency Name and Address Texas Department of Transportation Research and Technology Transfer Section/Construction Division P.O. Box 5080 Austin, TX 78763-5080		13. Type of Report and Period Covered Research Report (9/1/98 to 8/31/99) 14. Sponsoring Agency Code
15. Supplementary Notes Project conducted in cooperation with the Federal Highway Administration.		

16. Abstract

Freeway ramp design speed criteria contained in current American Association of Highway and Transportation Official (AASHTO) and Texas Department of Transportation (TxDOT) design policies have been traced through roughly 50 years of technical literature. The evolution of design speed criteria has been documented and technical rational leading to periodic changes has been included. TxDOT ramp design speed criteria are, essentially, the AASHTO criteria. The origins of driver acceleration and deceleration rates, which are built into the AASHTO criteria, are experimental studies performed during the late thirties. Several studies have raised questions about the appropriateness of the AASHTO minimum allowable ramp design speed, which is 50 percent of the freeway design speed. Questions have also been raised about the adequacy of high-speed ramp lengths designed by AASHTO criteria. A conceptual data collection plan has been designed to provide information that will answer questions regarding current criteria. Additionally, a nationwide survey of ramp design agencies indicates that (1) there is a variety of different design policies; (2) most designers have concern for entrance ramps, as opposed to exits; and (3) safety is the most commonly used evaluation measure.

17. Key Words: Design speed, ramp speed, interchange and ramp types, accidents, entrance ramps, exit ramps, sight distance, deceleration lanes, acceleration lanes, ramp design speed criteria	18. Distribution Statement No restrictions. This document is available to the public through the National Technical Information Service, Springfield, Virginia 22161.		
19. Security Classif. (of report) Unclassified	20. Security Classif. (of this page) Unclassified	21. No. of pages 104	22. Price

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REEVALUATION OF RAMP DESIGN SPEED CRITERIA

by

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Research Report Number 1732-2

Research Project Number 0-1732
Project Title: *Reevaluation of Ramp Design Speed Criteria*

Conducted for the

TEXAS DEPARTMENT OF TRANSPORTATION

in cooperation with the

U.S. DEPARTMENT OF TRANSPORTATION
Federal Highway Administration

by the

CENTER FOR TRANSPORTATION RESEARCH
Bureau of Engineering Research
THE UNIVERSITY OF TEXAS AT AUSTIN

December 1999

ABSTRACT

Current freeway entry ramp design speed criteria were evaluated through observations of twenty ramps in four Texas cities. Field observations of ramp and freeway traffic speed-distance relationships were made using videotaping methods. Traffic operations were described in terms of ramp and freeway right-lane speeds and accelerations, as well as ramp driver merging locations, accepted time gap sizes, and freeway time headways. The researchers determined that observed ramp driver acceleration rates and AASHTO values were comparable. For virtually all observations, ramp driver speeds are found to be greater than 50 percent of the freeway design speed, leading to a recommendation that the design policy provision allowing ramp design speeds to be as small as 50 percent of the freeway design speed be deleted. The ability of entry ramp drivers to see, prior to reaching the ramp gore, freeway right-lane traffic, into which merging is intended, was found to be very important. This finding led to a recommendation that the AASHTO acceleration lane length measurement model, for taper type ramps, be modified. The acceleration lane should be considered to begin only when ramp drivers have an unobstructed view of freeway right-lane traffic.

DISCLAIMERS

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ACKNOWLEDGMENTS

The authors express their sincere thanks for the support of TxDOT Project Director Ed Pensock (Texas Turnpike Authority Division) and Program Coordinator A. J. Krejci (DES). The authors also acknowledge the video support provided by personnel at the Houston Transtar and San Antonio Transguide traffic operations centers.

Research performed in cooperation with the Texas Department of Transportation and the U.S. Department of Transportation, Federal Highway Administration.

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CHAPTER 1. BACKGROUND AND OBJECTIVES

PROBLEM DESCRIPTION

Ramp facilities provide all freeway entrance and exit opportunities. Freeway sections adjacent to ramps are, therefore, analogous to arterial street at-grade intersections in that they create traffic-stream friction that limits freeway speed and capacity. Freeway bottlenecks most often develop in the vicinity of entrance and or exit ramp junctions. Clearly, excellent freeway ramp design is a critical freeway operational consideration. Currently, AASHTO [1] and TxDOT [3] design procedures state that ramp design speeds are to be a percentage of the freeway design speed. The TxDOT design procedures manual [3] states

> All ramps and connections shall be designed to enable vehicles to leave and enter the traveled way of the freeway at no less than 50% (70% usual, 85% desirable) of the freeway's design speed. . .

The choice of ramp design speed can significantly affect ramp curve radii, stopping sight distances, and speed-change lane lengths. Changes in vehicle characteristics and driver behavior may influence the predicted vehicle performance upon which the ramp design speed policy is based. Also, many of today's ramp designs may not fall within the scope of the original studies upon which the designs are being based. The need exists to thoroughly evaluate not only current freeway ramp design speed policy, but also related ramp facility elements, including acceleration or deceleration lane lengths.

OVERVIEW

The purpose of Chapter 1 is to review the current TxDOT and AASHTO design standards and to provide a historical perspective on their development. To achieve this goal, this section first reviews the evolution in design that has led to the current standards; it then presents these design standards as found in the TxDOT *Transportation Highway Design Division Operations and Procedures Manual* [3] and the AASHTO *Policy on Geometric Design of Highways and Streets* [1]. The concepts discussed apply to ramp design, but some of the minimums and assumptions given in the following are for "open road conditions." In general, these minimums also apply to ramp design, but any differences will be highlighted.

TERMINOLOGY

In reviewing current design standards and their evolution, this report uses the following concepts: design speed, safe stopping-sight distance, horizontal curvature, vertical curvature, ramp, ramp terminal, and speed-change lanes. These terms are defined below.

Design Speed

> *Design speed is the maximum safe speed that can be maintained over a specified section of highway when the design features of the highway govern. All facilities should be designed with all elements in balance, consistent with an appropriate design speed. Design elements such as sight distance, vertical and horizontal alignment, lane widths, roadside clearance, superelevation, etc. are influenced by design speed. It is therefore important that an appropriate design speed be selected.*
>
> TxDOT *Operations and Procedures Manual* [3]

Design speed is generally indicative of the type of operation expected on a facility. Freeways typically have design speeds ranging between 60 mph (97 km/h) and 80 mph (128 km/h); lower-level facilities, such as arterials and collectors, have lower design speeds of approximately 30 mph (48 km/h) to 60 mph (97 km/h).

Safe Stopping-Sight Distance

Safe stopping-sight distance is the minimum roadway distance visible to the driver required to provide adequate distance to react and to stop the vehicle. Sight distance should provide the driver sufficient time to gather information, process it, perform the required control actions, factor in the vehicle's response time, and evaluate the appropriateness of possible responses [11]. Stopping-sight distance is typically considered to be a sum of two distances: the distance traveled from the instant the driver sees the object to the instant the driver applies the brakes (PIJR, or perception, identification, judgment, and reaction), plus the distance the vehicle travels during braking.

Horizontal Curves

A horizontal curve is one of the two primary types of curves (horizontal and vertical). The horizontal curve standards represent the maximum degree of curve, or minimum radius. Horizontal curve design is based on a general relationship among superelevation (inclination of the roadway towards the center of curve), side friction factor (represents radial force caused by the friction effect between the tires and the roadway), vehicle speed, and curve radius.

Vertical Curves

The vertical curve is used wherever a change in elevation must be achieved. For example, a vertical curve may be used to connect two portions of a roadway at different elevations. This situation often occurs on ramps where the connection of the ramp on one road is at a different elevation than the ramp connection to the other road. The predominate factors affecting the safe design of a vertical curve are adequate sight distance, comfort, drainage control, general appearance, and headlight sight distance.

Ramp

For this report, the term *ramp* is defined in accordance with the definition provided in the AASHTO design guide, which states that "the term 'ramp' includes all types, arrangements, and sizes of turning roadways that connect two or more legs at an interchange. The components of a ramp are a terminal at each leg and a connecting road, usually with some curvature, and on a grade" [1]. The connecting roadway is often referred to as the *ramp proper*. This definition differs slightly from that given in the TxDOT operations and procedures manual, which defines a ramp to be what AASHTO refers to as the *ramp terminal* and the portion of roadway connecting to the ramp terminals as *connecting roadways.*

Ramp Terminal

The ramp terminal is the portion of the ramp adjacent to the through-travel way.

Speed-Change Lanes

This report adopts the AASHTO definition of speed-change lane, which is "the added pavement joining the travel way of the highway with that of the turning roadway and does not necessarily imply a definite lane of uniform width." The speed-change lane is commonly referred to as the acceleration or deceleration lane.

DEVELOPMENT OF CURRENT RAMP DESIGN STANDARDS

By reviewing the developments leading to today's TxDOT and AASHTO design standards, one hopes to gain insight into the current design standards' applicability to today's traffic. Unfortunately, neither the 1988 TxDOT manual nor the 1990 AASHTO guide provides much insight into the reasoning behind the recommended ramp design standards. A better source for information about the origin of the current design standards is the 1965 *Policy on Geometric Design of Rural Highways* [4], by the American Association of State Highway Officials (AASHO, which has evolved into the present-day AASHTO). The guide is one in a series of continuing updates that have led to the publication of the most recent AASHTO and TxDOT design manuals. To distinguish between the design manuals, the above guide will be referred to as the 1965 AASHO guide, while the 1990 AASHTO design guide manual will continue to be referred to as the AASHTO design guide or 1990 AASHTO guide.

Design Speed

One of the most fundamental parameters affecting a design is the design speed. Examination of AASHTO and TxDOT design standards shows that once a design speed is chosen, the critical speed for determining actual design features (e.g., lane lengths and curve radii) is the assumed average running speed of vehicles for that design speed. The average running speeds used in the 1988 TxDOT manual and the 1994 AASHTO manual are first seen in the 1965 AASHTO guide, in which these speeds are apparently based on studies from the 1950s and 1960s. The earlier 1954 AASHO guide suggests running speeds that are significantly lower than those of the 1965 guide.

Some doubt exists as to whether these assumed average running speeds are still accurate under today's conditions. One study, the 1992 *Speed Estimates for Roadway Design and Traffic Control* [19], suggests that the speed estimates used are significantly below the actual speeds. One estimate suggested that as much as 90 percent of observed traffic was exceeding posted speed limits, which are often near the assumed low-volume running speed. Additional support for a growing belief that the assumed average running speeds are unrealistic may be found in the methodology described in the final report for NCHRP 3-35 [7]. In this study, the speed used to determine the required length of a speed-change lane was the design speed itself, not the AASHTO running speed.

Guide Values for Ramp Design Speed

One of the earliest recommendations for design speed of ramps may be found in *Proposed Design Standards for Interregional Highways*, 1944 [18]. This document recommended that "all ramps and connections would be designed to enable vehicles to leave and enter the highway at 0.7 of the highway's design speed." Over time, changes have been made to this recommendation — changes that are reflected in Table 1.3. This table first appeared in its current form in the 1984 AASHTO manual. Previously, the recommendations had been slightly different; AASHTO had only had guidelines for desirable and minimum ramp design speeds, rather than the three ranges (upper, middle, and lower) seen today. While the upper or desirable recommended ramp design speed is approximately the average low volume running speed for the freeway design speed, the minimum and middle ranges do not appear to have a correlation with some traffic characteristic or design parameter. No literature has been found which reveals the source of these recommendations or reason for the changes in each subsequent AASHO/AASHTO design manual update.

Design Speed-Change Lanes

The 1954 and 1965 AASHO guides' presentations of the design of speed-change lanes provide more information than later design guides on design standards derivation. The 1965 manual defines a speed-change lane as "...an auxiliary lane, including tapered areas, primarily for the acceleration or deceleration of vehicles entering or leaving the through traffic lanes. The term speed-change lane, deceleration lane, or acceleration lane, as used herein, applies broadly to the added pavement joining the traveled way of the highway with that of the turning roadway and does not necessarily imply a definite lane of uniform width."

An examination of the 1965 AASHO guide leads to the conclusion that most design values are the same as those in the 1990 AASHTO guide, with differences lying in implementation. Interestingly, it will be shown that many of the design values found in the 1988 TxDOT design guide are also included in the 1965 AASHO guide, whereas a simple comparison of the 1990 AASHTO design guide and the 1988 TxDOT guide may lead one to believe that certain design values were developed separately. To demonstrate and provide insight into both the current TxDOT and AASHTO design standards, this report will present the rationale behind the design of the taper section and deceleration and acceleration lane lengths.

Taper Section. One of the first design aspects covered in the speed-change lane design is taper. This is the taper at the end (or beginning) of the speed-change lane, which is not to be confused with the taper-type speed-change lane design. Although the current AASHTO design recommends fixed taper lengths, the 1965 AASHO guide recommended variable taper lengths. (These were based upon passing practices on two lane highways, as determined in a 1941 study [6].) It appears that the current fixed AASHTO taper lengths are a simplification of the 1965 variable taper lengths, utilizing the longer taper values for all design speeds. The source for the TxDOT taper length will be discussed in the acceleration lane section.

Deceleration Lanes. The 1965 AASHO guide bases the deceleration lane length on three factors: "(a) the speed at which drivers maneuver onto the auxiliary lane; (b) the speed at which drivers turn after traversing the deceleration lane; and (c) the manner of decelerating or the deceleration factors."

The first factor is based upon the assumption that, when shifting into the deceleration lane, most drivers travel at a speed no greater than that of the low volume average running speed. The second factor is assumed to be the running speed of the sharp or controlling curve of the ramp proper. The third factor is based on general observations and several limited studies. The design values for many of these factors in the current design standards are the same as those in the 1965 guide; many are based on several studies [5, 8, and 9] conducted primarily in the late 1930s.

As a final point, it should be noted that this design is based on passenger vehicle operation. While the 1965 AASHO guide recognizes that trucks require a longer deceleration distance for the same speed differential, it is assumed that "longer lanes are not justified because average speeds of trucks are generally less than those of passenger cars."

Using the factors discussed above, the 1965 AASHO manual develops a table of lengths of deceleration lanes for various combinations of highway and ramp design speeds. A comparison of this table and the deceleration length table in AASHTO reveals these tables to have exactly the same design values. The only numerical difference is that the 1965 version gives design values for highway design speeds of 75 mph (120 km/h) and 80 mph (128 km/h), whereas the 1990 AASHTO version gives design values only as high as 70 mph (112 km/h). However, a critical difference does exist between the methods of applying the two guides' design lengths. The 1965 AASHO guide assumes the taper to be part of the total speed-change lane length, whereas the 1990 AASHTO guide treats the taper as additional length. This means that while not altering any of the fundamental data on which the lengths are derived, i.e., average running speeds, acceleration and deceleration rates, etc., the 1990 AASHTO guide deceleration lanes are essentially 300 ft (91 m) (a typical taper length) longer than the 1965 AASHO lengths. The primary differing assumption between the two manuals seems to be that the 1965 AASHO guide assumes that deceleration occurs while the vehicle shifts from the freeway lane to the deceleration lane, while the 1990 AASHTO guide assumes deceleration does not begin until the vehicle has completely entered the deceleration lane. This change in assumptions appears to have occurred between the 1965 AASHO guide (the blue book) and the 1973 AASHTO guide [10] (the red book). No literature has been discovered that explains the rationale behind this change; the red book actually refers the reader to the blue book for an explanation of how the speed-change lane design values are derived.

The 1965 AASHO blue book also includes a table of deceleration length values rounded to the nearest 25 ft (7.6 m) increment. These were the values used in the determination of speed-change lane lengths for design. This rounded version of the deceleration lane length for design was omitted in subsequent versions of the AASHO/AASHTO guide (1973 and 1984), with the later versions using the raw numbers as still found in the current AASHTO guide. Interestingly, the deceleration length table in the current TxDOT manual is exactly the same as the rounded version of the AASHO 1965 guide design lengths. This realization provides an important link between the TxDOT and AASHTO guides, a link that is not readily realized in the comparison of the latest versions of these guides; that is, the TxDOT values are based on the same studies and methodology as the current AASHTO values. There has been no literature found to explain why AASHTO ceased using the rounded table or why TxDOT did not change to the raw design lengths along with AASHTO.

Acceleration Lanes. With respect to both acceleration and deceleration lanes, the 1965 AASHO guide offers little guidance between the parallel and taper-type speed-change lanes. There is a sense of an unstated assumption that most acceleration lanes will be of the taper type (at a 50:1 taper) with only a brief mention that some designers may prefer a parallel acceleration lane with a more acute taper at the end. The recommended 50:1 taper is readily seen as that carried into the TxDOT standard designs and the 1990 AASHTO taper-type designs, with the exception of AASHTO recommending a range from 50:1 to 70:1.

The 1965 AASHO guide bases the length of the acceleration lane on several factors; "(a) the speed at which drivers merge with through traffic; (b) the speed at which drivers enter the acceleration lanes; and (c) the manner of accelerating or the acceleration factors... and may depend on the relative volumes of the through and entering traffic." Many of the rationales and assumptions used are similar to those about the deceleration lanes. For example, as with deceleration rates, the manner of acceleration or the acceleration rates are determined from studies predominately completed in the late 1930s, and the design lengths are based on passenger-car characteristics. These studies produced both estimates of maximum and normal acceleration rates that underlie the length values still in use today.

For factors (a) and (b), the 1965 AASHO guide states that satisfactory merging behavior would be achieved by a vehicle in the acceleration lane entering the freeway through-lane at a speed 5 mph (8 km/h) lower than the average freeway running speed. Also, this vehicle is assumed to enter the acceleration lane at a speed equal to the controlling speed of the ramp proper. Therefore, the speed differential for determining the length of the acceleration lane is the difference between the average running speed of the freeway, less 5 mph (8 km/h), and the average running speed of the controlling curve on the ramp proper. It has been suggested that the 5 mph (8 km/h) incremental difference assumed by AASHO may not hold. For example, it is possible that drivers do not merge in response to some threshold speed differential but instead that they will merge at any speed differential, with their merging dependent on some other element, such as vehicular angular velocity [7].

Similar to the deceleration lane lengths table, an acceleration lane lengths table was produced in the 1965 guide for various combinations of freeway and ramp design speeds. This table is exactly the same as the table for the determination of acceleration lanes in the 1990 AASHTO guide (Table 1.2 of this report). The 1965 AASHO manual also provided a rounded set of design values that was dropped in later design manuals. Finally, as with the deceleration lanes, the 1965 AASHO design considers the taper to be part of the speed-change lane length. Therefore, while design values are the same numerically in the 1965 and 1990 manuals, the 1990 measurement method will result in longer acceleration lane lengths.

Once again, it is possible to connect the 1965 AASHO guide directly to both the 1990 AASHTO guide and the 1988 TxDOT guide. It has already been observed that the acceleration lane lengths from the 1965 AASHO guide are utilized in the 1990 AASHTO guide. It is logical, then, to assume that the situation will be similar to that of the deceleration lanes, and that there is a connection between the 1988 TxDOT design guide acceleration lane length and the earlier AASHO work. An examination of TxDOT's standard designs reveals that all TxDOT acceleration lanes, for single-lane ramps, are designed in the same general manner, utilizing a taper-type design. Compared to the 1965 rounded acceleration length table, the current TxDOT design satisfies all design lengths for freeway design speeds of 50 mph (80 km/h) and 60 mph (96 km/h), and it falls between ramp design speeds of 30 mph (48 km/h) and 35 mph (56 km/h) for a 70 mph (112 km/h) freeway design speed. The fact that the minimal design speed of a ramp is to be 50 percent of the freeway design speed, i.e., 35 mph (56 km/h) for a 70 mph (112 km/h) freeway, supports the assumption that the TxDOT standard design is based on the 1965 AASHO methodology. It would seem that TxDOT design officials decided to utilize one standard design that satisfied all acceptable ramp/freeway design speed combinations. Accordingly, for any situation other than the 70 mph (112 km/h) freeway and 35 mph (56 km/h) ramp (the maximum acceleration lane length case), TxDOT's design would be conservative, according to the 1965 methodology, and would use a longer length than AASHO recommended.

A clarification of the taper section can connect the TxDOT standard design to the 1965 AASHO methodology even further. As noted earlier, the 1965 AASHO guide utilized varying lengths of taper sections, but for the taper-type acceleration lane design the guide recommends a 50:1 taper for the speed-change lane. Therefore, at the end of the acceleration lane, the taper section in which the lane is reduced from 12 ft (3.7 m) wide to 0 ft wide would be equal to 12 ft (3.7 m) multiplied by 50, or 600 ft (183 m) — exactly what is used in the TxDOT design. This is roughly double the taper length used in a parallel-type design.

Acceleration and Deceleration Rates. As noted, the values used for acceleration and deceleration rates directly influence the speed-change lane length. An extensive discussion of the 1930s studies upon which the TxDOT and AASHTO values are based can be found in *Reevaluation of Ramp Design Speed Criteria: Review of Practice and Data Collection Plan* [24]. This report clearly demonstrates uncertainty about the actual acceleration rates that would be best suited for the design. It would appear that even though the rates from the 1930s may be lower than those used by drivers today, deficiencies in the AASHTO speed-change lane model (i.e., gap acceptance) may require these conservative rates to assure adequate lengths. That is, unrealistically low speed-change lane lengths most likely would result from updating acceleration and deceleration rates and utilizing the current AASHTO methodology. While it is possible that the AASHTO lengths and design standards used are acceptable, studying ramp operations and determining if adequate designs are being implemented is certainly justified. There are at least three outcomes possible from a review of the speed-change lane design: (1) the designs are too conservative (the 1938 acceleration/deceleration rates are to conservative) and shorter lengths may be justifiable; (2) the designs are acceptable (the 1938 rates properly compensate for model deficiencies); and (3) the designs are inadequate (the 1938 rates do not adequately compensate for model deficiencies).

RAMP DESIGN

The "current practices" discussed in this report are based on the TxDOT Highway Design Division *Operations and Procedures Manual* [3] and the AASHTO *A Policy on Geometric Design of Highways and Streets, 1990* [1]. The effect of the choice of ramp design speed on the various geometric features and operational characteristics will be seen throughout this section. Some concerns about potential difficulties in the current design standards will also be raised.

Design Speed

The primary focus of this study is the current relationship between the design speed of the intersecting highway and the choice of ramp design speed. Current TxDOT guidelines state that "All ramps and connections shall be designed to leave and enter the traveled way of the freeway at no less than 50% (70% usual, 85% desirable) of a freeway's design speed." Table 1.1 is referenced from the AASHTO design guide and reflects the design guide values for ramp design speed and highway design speed. Figure 4-54 in the TxDOT design guide reproduces the design values for 50, 60, and 70 mph (80, 97, and 112 km/h).

According to AASHTO, ramp design speeds should approximate the low-volume running speeds on intersecting highways. Where this design speed is not practical, ramps should not be designed at less than 50% the design guidelines. For freeway and expressway ramps, only those values of highway design speed above 50 mph (80 km/h) apply.

Table 1.1 Guide Values for Ramp Design Speed as Related to Highway Design Speed

Highway Design Speed (mph)	30	40	50	60	65	70
Ramp Design Speed (mph)						
Upper Range (85%)	25	35	45	50	55	60
Middle Range (70%)	20	30	35	45	45	50
Lower Range (50%)	15	20	25	30	30	35

Corresponding Minimum Radius (ft), see Table III-6; Source: 1990, AASHTO, Table X-1, Page 960

These design values are considered to apply to the sharpest or the controlling ramp curve. This curve will usually be on the ramp proper, that is, on the connecting roadway between the two ramp terminals. These design speeds are not considered to apply to the ramp terminals since the ramp terminals should be provided with speed-change facilities adequate for the highway speed involved. A discussion of the design of ramp terminals is provided in subsequent sections.

The following is a short summary of AASHTO recommended guidelines for considering design speed on the various ramp types.

Diagonal Ramps. A value in the middle range is often practical.

Loops. Minimum values usually control each design, although the loop design speed should not be less than 25 mph (40 km/h) for highway design speed over 50 mph (80 km/h).

Semidirect Connections. Middle and upper ranges should be used with a minimum acceptable design speed of 30 mph (48 km/h). Typically, it is not practical to utilize a design speed greater than 50 mph (80 km/h) for short, single-lane ramps.

Direct Connections. Middle and upper ranges have a desirable minimum design speed of 40 mph (64 km/h). The minimum design speed is not to be less than 35 mph (56 km/h) in any case.

For situations in which a ramp connects two intersecting highways, the ramp design speed is to be based on the highway with the higher design speed. However, it may be acceptable to vary the design speed, with the portion of the ramp closer to the higher design speed highway based on the higher speed, and the portion of the ramp closer to the lower design speed highway based on the lower speed. Where the ramp is used to connect a freeway to a major crossroad or street, forming an at-grade intersection where signal or sign control may be in effect, the design of that portion of the ramp at the crossroad is based on intersection design controls.

Superelevation (e) and Side Friction Factor (f)

AASHTO has established limiting values of *e* and *f* for different design speeds on open roadways. The maximum superelevation and side friction factors are constrained by practical limitations. These limits are affected by items such as climate conditions (if an area is subject to ice and snow), pavement conditions, pavement types, increased potential for hydroplaning, area type (urban or rural), terrain conditions, driver discomfort at low or high speeds, and trucks with higher centers of gravity. Based on studies and experience, the maximum rate of superelevation on highways is typically 0.10, and occasionally 0.12. In areas subject to ice and snow 0.08 provides a practical limiting value. Side friction factors for design purposes range from .17 for 19 mph (30 km/h) to .09 for 75 mph (120 km/h) [1].

TxDOT's *Operations and Procedures Manual* provides tables relating the usual and absolute maximum degree of curve (minimum radius) for design speeds of 30, 40, 50, 60, and 70 mph (48, 64, 81, 97, and 112 km/h) and a superelevation of 0.08. The absolute maximums are based directly on those maximums calculated in AASHTO. The TxDOT guide refers the designer to the AASHTO design guide for maximum degree of curve (minimum radius) values that apply to superelevation rates other than 0.08. The TxDOT manual also provides the superelevation to be used for various design speeds and degrees of curvature where the limiting values are not utilized. These values are also based on a maximum superelevation of 0.08.

While the methodology for ramp curves does not change from that of open road horizontal curve, there are some disagreements in the limiting values of the *e* and *f*.

Concerns have been raised with the AASHTO guide and, subsequently, the TxDOT guide [13]. Different interpretations of how the minimum ramp design should be implemented exist. In this report these areas of contradicting interpretations are not directly addressed, as only existing designs are studied and design standards are assumed to follow the typical TxDOT design layouts listed in a later section of this report. The thrust of this study is a comparison of low-speed ramp design standards as a general category with that of higher-speed ramp design standards. A complete discussion of the varying interpretations and their impacts on design may be found in *Reevaluation of Ramp Design Speed Criteria: Review of Practice and Data Collection Plan* [24].

Sight Distance

Sight distances along ramps should be at least as great as the safe stopping sight distance. Sight distance is addressed in the TxDOT *Operations and Procedures Design Guide* as follows [3]:

> *On all ramps and direct connections, the combinations of grade, vertical curves, alignments and clearance of lateral and corner obstructions to vision shall be such as to provide sight distance along such ramps and connections from terminal junctions along the freeway, consistent with the probable speeds of vehicle operation.*

Within the ramp design section, the TxDOT guide provides a table for minimum stopping sight distance and desirable stopping sight distance for various design speeds. These stopping sight distances are identical to those found in AASHTO. Additional considerations from AASHTO include that the freeway preceding an exit ramp should have a sight distance for through-traffic that is based on the highway design speed and that exceeds the minimum stopping sight distance by at least 25 percent [1].

Concerns about the current stopping sight distance criteria have been raised in various studies [15, 24]. Issues include insufficient break reaction time for the elderly, potential insufficient design for trucks, and side friction factors that do not account for the greater demand caused by curves.

Grades

A ramp will typically consist of a central portion with a high grade while the ramp terminals will be of lesser grades. The limiting gradient of this central portion of the ramp is influenced by the effect of the steepness and length of the grade on vehicle operations, and by the need to provide adequate sight distance. The ramp design speed will be predominant in both these factors. The general AASHTO guidelines for ramp gradients follow an expectation that higher ramp design speeds will have flatter gradients. The AASHTO general criteria are as follow:

> *…it is desirable that ascending gradients on ramps with a design speed of 70-80 km/h be limited to 3 to 5 percent; those for 60 km/h speed, to 4 to 6 percent; those for a 40 to 50 km/h speed, to 5 to 7 percent; and those for a 30 to 40 km/h speed, to 6 to 8 percent. Where topographic conditions dictate, grades steeper than desirable may be used. One-way descending gradients on ramps should be held to the same general maximums, but in special cases they may be 2 percent greater.* [1]

The ramp terminal grades are largely determined by the through-road profiles.

The TxDOT standards differ slightly from the AASHTO discussion. TxDOT utilizes the same vertical curve relationships as AASHTO, although it incorporates a minimum length for crest vertical curve. This results in shorter minimum sag vertical curves at speeds below a 40 mph (64 km/h) design speed and longer minimum sag vertical curves at speeds above 40 mph (64 km/h). The TxDOT guide also states that the "tangent or controlling grade on ramps should be as flat as possible, and preferably should be limited to 4 percent or less." This does not account for differing design speeds as does the AASHTO manual, creating a more conservative standard.

Other Ramp Design Issues

In general, ramps should be designed as single-lane facilities with provision for emergency parking, although where the capacity of a one-lane ramp is not sufficient a two-lane facility may be provided. Also, right-hand ramps are considered superior to left-hand ramps in operation and safety characteristics.

While not discussed in this report (since it is not critical to acceptable minimum design), a ramp design will probably require superelevation runoff through a superelevation transition. If the minimum ramp length does not provide adequate length for this superelevation transition, then the ramp will require lengthening, or the design speed and superelevation chosen will need to be revisited. Loop ramps are an example of how superelevation must typically be developed into and out of the ramp proper. Also not discussed in this report are ramp gore design and pavement widths. While related to highway and ramp design speeds and impacted by the type of ramp and ramp design, the effect of gore design and ramp pavement width is not critical to the ramp design questions under study. For an in-depth review of these topics the reader is directed to the TxDOT manual and the AASHTO design guide.

Ramp Terminals

There are two distinctive operating scenarios for ramp terminals. A ramp terminal may be free flow, with traffic merging or diverging at flat angles (e.g., ramps adjacent to a freeway); or the ramp may terminate to a minor road (e.g., a cloverleaf ramp into the crossroad at an interchange). The area of interest for this report and, therefore, for the design discussion that follows is for the free flow ramps. Minimal attention is given to exit ramps at this time since a primary focus of this study is entrance ramp operations. Discussion of the nonfree flow ramp terminal and exit ramps may be found in the TxDOT and AASHTO design guides and *Reevaluation of Ramp Design Speed Criteria: Review of Practice and Data Collection Plan* [24].

General. Ramp terminals must be designed to account for sight distance and the design of the ramp proper. The AASHTO manual presents a concise example of some important considerations as follows:

> *Profiles of ramp terminals should be designed in association with horizontal curves to avoid sight restrictions that will adversely affect operations. …At an entrance terminal from a ramp on a ascending grade, the portion of the ramp and its terminal intended for acceleration should closely parallel the through-lane profile to permit entering drivers to have a clear view ahead, to the side, and to the rear on the through road.* [1]

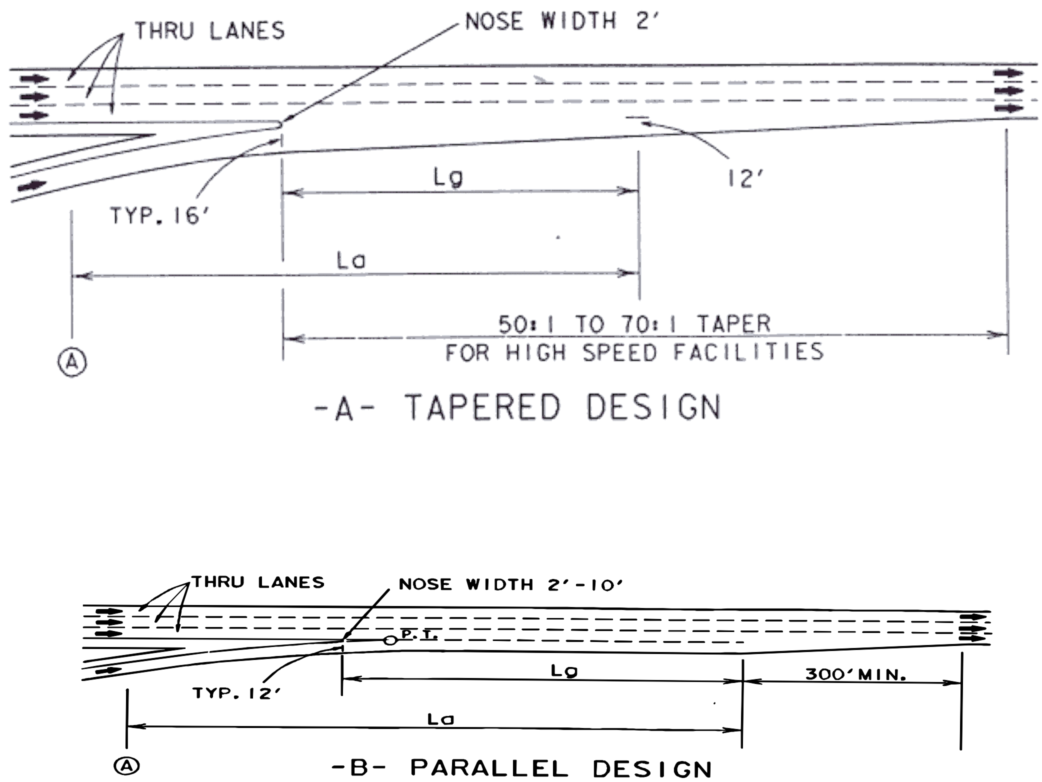
Desirably, ramp terminals are placed before the interchange and on the right side of the freeway. Adequate sight distance must be provided on the freeway before the ramp terminal to allow for decision making and maneuvering. Also, consideration must be given to the ramp terminal placement concerning the distance between the free flow terminal and the structure. Typically, the distance required between a ramp preceding the interchange structure and the structure is not as great as the distance required between the structure and a ramp terminal on the far side.

Speed-Change Lanes. The speed change lane is a critical portion of any ramp terminal design. It is within the speed-change lanes that entering motorists accelerate to a speed adequate for merging with through traffic. The speed-change lane should be sufficiently long to enable a driver to change speed, in a safe and comfortable manner, from the ramp speed to the highway speed. A primary consideration, stated in the AASHTO design guide, for acceleration-lane length is the need for sufficient length to permit speed adjustments of both the through and entering vehicles so that entering vehicles may find and maneuver into a gap before the acceleration lane. A later section of this report will provide an in-depth review of freeway gap acceptance and merging.

There are two basic designs of freeway ramp terminals and speed-change lanes: taper and parallel. The taper type involves direct entry or exit of the vehicle at a flat angle, and the parallel type utilizes an added lane for speed changes. In theory, the taper type fits well with drivers' desired paths and reduces the amount of steering control necessary, although it requires the driver to time-share between tasks of accelerating, gap search, and steering [14]. Figure 1.1, taken from the AASHTO design guide, illustrates both design types for single-lane entrances.

Taper-Type Entrances. When properly designed, the taper entrance is considered able to function smoothly at all volumes, including the design capacity of the merging area. The AASHTO design guide recommends that the entrance ramp be brought into the freeway at a rate of 50:1 to 70:1, between the outer edge of the acceleration lane and the inside edge of the freeway. The desire of the AASHTO standards is to create a taper-type design such that a vehicle may reach a speed approximately 5 mph (8 km/h) lower than the average highway running speed by the point at which the left edge of the ramp meets the right edge of the travel way. For consistency AASHTO sets this point to be where the right edge of the ramp and travel way are 12 ft (3.7 m) apart.

The length required for a vehicle to achieve a speed 5 mph (8 km/h) below the average running speed is referred to as the *acceleration length,* L_a, by AASHTO and is shown in Figure 1.1. This length is typically measured from the end of the governing curve on the ramp proper to where the right edge of the ramp proper and through lane are 12 ft (3.7 m) apart. This distance is based on the speed differential between the average running speed on the curve entrance and the highway. Table 1.2 (AASHTO Table X-4) gives the value of this distance for various curve design speed and highway design speed combinations. In addition to the minimum acceleration length, the AASHTO design guide requires a check to see that a minimum gap acceptance length is met (see Figure 1.1). Adjustments are also provided for the existence of grades, lengthening L_a on upgrades and decreasing L_a on downgrades.



Notes:

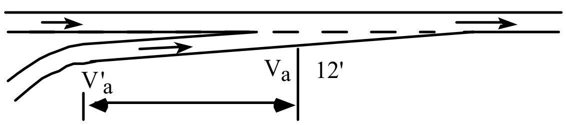
1. L_a is the required acceleration length as shown in Table X-4 or X-5.
2. Point (A) controls safe speed on the ramp. L_a should not start back on the curvature of the ramp unless the radius equals 1,000 ft or more.
3. L_g is required Gap Acceptance Length. L_g should be a minimum of 300 ft to 500 ft depending on the nose width.
4. The value of L_a or L_g, whichever produces the greatest distance downstream from where the nose width equals 2 feet, is suggested for use in the design.

Figure 1.1. Tapered and Parallel Entrance Ramp Designs [1]

Table 1.2 Minimum Acceleration Lengths for Entrance Terminals with Flat Grades of 2 Percent or Less

			Acceleration Length, L (ft) for Entrance Curve Design Speed (mph)								
		Stop Condition	**15**	**20**	**25**	**30**	**35**	**40**	**45**	**50**
Highway Design Speed	**Speed Reached, V_a**		**and Initial Speed, V'_a (mph)**							
(mph)	**(mph)**	**0**	**14**	**18**	**22**	**26**	**30**	**36**	**40**	**44**
30	23	190	----	----	----	----	----	----	----	----
40	31	380	320	250	220	140	----	----	----	----
50	39	760	700	630	580	500	380	160	----	----
60	47	1,170	1,120	1,070	1,000	910	800	590	400	170
70	53	1,590	1,540	1,500	1,410	1,330	1,230	1,010	830	580

Source: 1990, AASHTO, Table X-4, Page 986.



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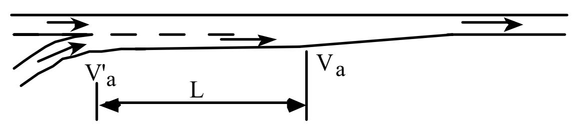


PARALLEL TYPE

Parallel-Type Entrance. On the parallel-type ramp, the vehicle is assumed to accelerate to the near-freeway speed necessary for merging on the parallel acceleration lane. At the end of the acceleration lane there is a taper to guide a vehicle onto the freeway through lanes. AASHTO recommends a 305 ft (93 m) taper for highway design speeds up to 70 mph (112 km/h). The difference between the two types of ramps (taper and parallel) is not the minimum acceleration length required, but the point from which it is measured. For the parallel type, entrance length of the acceleration lane is measured from the point where the left edge of the ramp meets the right edge of the freeway to the beginning of the taper. That is, acceleration on the parallel-type ramp occurs in the lane parallel to the freeway through lanes, downstream from the point of convergence of the freeway and ramp, whereas acceleration on the taper-type ramp occurs on the ramp proper, upstream from the point of convergence of the two roadways. An exception to this may occur where a parallel-type ramp has a large radius upstream of the convergence point, and the motorist's view of the freeway while on the ramp is unobstructed. Under these conditions part of the ramp proper may be used as part of the acceleration length. Where the freeway and ramp are anticipated to carry volumes approximating the design capacity of the merging area, AASHTO recommends a minimum length of at least 1,275 ft (373 m) plus taper. The preceding Figure 1.1 illustrates a typical parallel-type entrance ramp terminal and the minimum acceleration distance for parallel type speed change as given by Table 1.2.

TxDOT Speed-Change Lane Design. The preceding discussion of speed-change lanes concentrated on the AASHTO approach to speed-change lane design. TxDOT has adopted standard designs that differ from this approach. Figures 1.2 (page 23) is an example of a TxDOT standard design from the 1988 TxDOT design guide.

Entrance Ramps. A review of the TxDOT standard ramp designs reveals that TxDOT recommends one standard taper-type entrance ramp design for all single-lane entrance ramps. This design consists of three sections: (1) a 246 ft (75 m) section upstream of the gore; (2) a 460 ft (140 m) section, at a 50:1 taper, downstream of the gore; and (3) a 614 ft (187 m) section, tapered at 50:1, which serves to reduce the acceleration lane width from 12 ft (3.7 m) to 0 ft (m). This design differs from the AASHTO in that only one standard speed-change lane length is utilized, whereas AASHTO utilizes varying speed-change lane lengths, according to Table 1.2, discussed previously.

A comparison of the TxDOT design to the AASHTO design shows that at lower design speeds the TxDOT design may provide less length than the AASHTO design. If the TxDOT acceleration lane length is measured based on the AASHTO methodology (i.e., not including the 12 ft [3.7 m] to 0 ft [m] width taper section), the provided acceleration lane length would be the sum of the first two sections — 705 ft (215 m). Compared to AASHTO, this length would be insufficient for a freeway design speed of 70 mph (112 km/h) and ramp design speeds of 45 mph (72 km/h) or less; a freeway design speed of 60 mph (96 km/h) and ramp design speeds of 35 mph (56 km/h), or less; or a freeway design speed of 50 mph (80 km/h) and ramp design speeds of 20 mph (32 km/h) or less. Therefore, the TxDOT design will not provide a sufficient length, compared to AASHTO, for 60 and 70 mph (96 and 112 km/h) freeways when the minimum ramp design speed of 50 percent the freeway design speed is utilized. If half of the third section — 305 ft (93 m) of the 610 ft (186 m) taper section of the TxDOT design — is included in the acceleration lane length, and this total length is compared to AASHTO, the TxDOT design would satisfy the minimum requirements for the 50 and 60 mph (80 and 96 km/h) freeway design speeds but not a 112 km/h freeway design speed. The entire taper length in the TxDOT design would need to be included to satisfy the required acceleration lane length for a 70 mph (112 km/h) freeway, according to AASHTO.

It should be noted that the preceding discussion, especially that of the TxDOT acceleration lane length being only 690 ft (210 m), is a worst-case scenario. According to the AASHTO standards, it may be possible to include more of the ramp length upstream of the TxDOT 246 ft (75 m) section as part of the acceleration lane length. The inclusion of more of the upstream length would be case specific, depending on the driver's ability to view the road not being hampered by grades, obstructions, and curvature. Clearly, some of the TxDOT designs would allow for inclusion of additional upstream length. Since the TxDOT standard design does not specifically address upstream design for all cases, it is not possible to make any general statement as to how much, if any, of the upstream ramp length should be included in satisfying the AASHTO recommended ramp acceleration lane lengths. Later in this report, both the TxDOT and AASHTO designs will be seen to be based on the same material.

Other TxDOT Ramp Operations Issues. TxDOT utilizes two different approaches to restricting access on a controlled-access facility. One method (H.B. 179 Planned Freeway) of access control is using State of Texas police power to control driveway access, subject to certain conditions. A second method (non-H.B. 179 Planned Freeway) controls access solely by provision of frontage roads. It was mentioned earlier that frontage road ramps may be used between interchanges or incorporated into interchanges. To avoid operational problems, possibly including blockage at the merge point of the ramp and frontage road due to queue storage, TxDOT has developed exit ramp to cross street separation distance requirements. This distance is based on accommodating weaving, braking, and traffic storage.

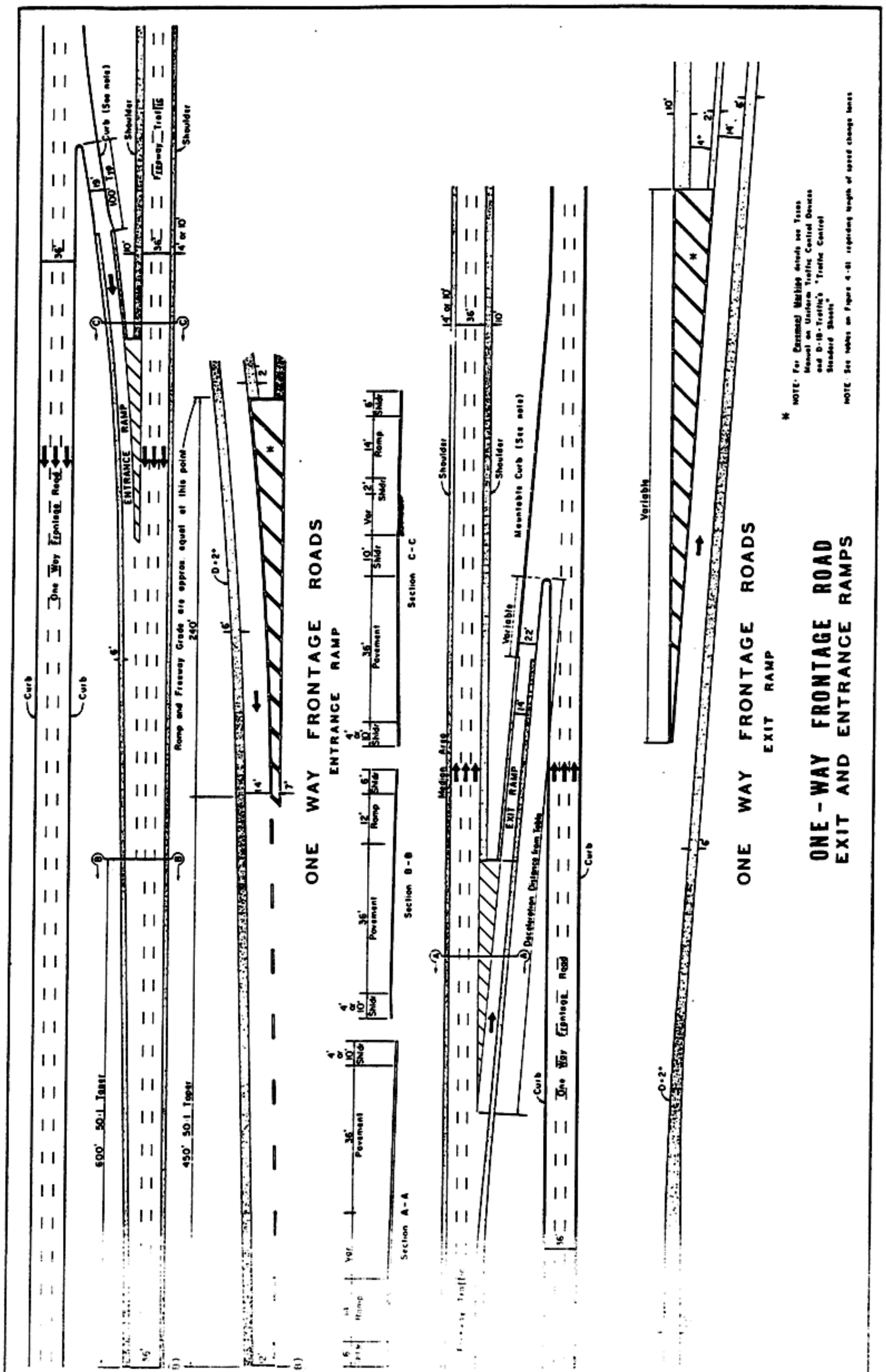


Figure 1.2 TxDOT One-Way Frontage Road Ramps [3]

ACCIDENTS

One aspect of ramp design, and geometric design in general, that has been sporadically studied over the past decades, is the effect of geometric features on accident rates. One of the most thorough and frequently referenced reports on the relationship between accidents and design is *Analysis and Modeling of Relationships between Accidents and the Geometric and Traffic Characteristics of the Interstate System* [16]. This study, based on data from twenty states, considered items such as number of lanes, design speed, lane width, maximum curvature, pavement type, grade, stopping sight distance, number of information and advertising signs, lighting, volume, and percent commercial vehicles. This study presented several findings of interest, including (1) that increased traffic volumes resulted in an increased number of accidents; (2) traffic-oriented variables (e.g., volume and percent trucks) contributed most to the variance in accidents on the Interstate system; (3) that geometrics alone accounted for only a small portion of the variance in accidents; and (4) that no relationship could be determined between the geometrics studied and fatalities.

The third and fourth points, involving geometrics, are clarified within the report. The sites chosen for study by each contributing state were supposedly "representative" locations rather than high-accident locations. This selection process probably eliminated from study those interchanges and ramps where under-designed geometric features would cause excessive accidents, though a small fraction of the data submitted did seem to have exceptionally high accident frequencies. For these interchanges a separate "failure analysis" was performed. This analysis better highlighted the effects of unusual design features, geometrics, and traffic characteristics not seen in the "representative" samples. While high traffic volumes are still regarded as the primary cause of accidents, additional information relating to geometrics can be obtained through failure analysis. This information includes the following: (1) that design speeds that are too low can, to a considerable degree, cause accidents, and (2) that on most of the ramp types, poor geometric features (short speed change lane, sharp curvature, and too short stopping-sight distance) can, to a considerable degree, cause accidents.

More recently, additional accident experience results were published in *Accidents and Safety Associated with Interchanges*, which provided a review of data and experience from other research efforts [12]. A brief synopsis of some of the conclusions of interest to this study were (1) an increase in accidents rates with increasing maximum curvature; (2) increasing accident rates with increasing average daily traffic; (3) horizontal curvature is a more significant factor in accident rates than grades; (4) both roll-over and skidding potential must be checked when designing ramp horizontal curves to accommodate trucks; and (5) the relative safety of an urban interchange is enhanced where 800 ft (244 m) or longer acceleration or auxiliary lanes are provided [12, 20, 21, 22, 23].

Other points have been raised in another study [13] in which the general conclusion was that current AASHTO design standards (specifically open road horizontal curve design parameters) provide for a safe operation, for both passenger cars and trucks. Accidents (rollover and skidding) were observed to occur at undesirable levels when unrealistic design speeds were used. Where AASHTO design assumptions (i.e., speed of vehicle) are not violated, adequate margins of safety are provided; but where vehicles exceed design speeds, unsafe conditions that can possibly lead to accidents may occur. This conclusion by this study highlights the need for careful selection of design speed and certainly leads one to at least question whether the AASHTO allowance of minimum ramp design speeds at 50 percent of the freeway design speed is reasonable.

RESULTS FROM OTHER MODELS

Researchers and agencies have attempted to refine the AASHTO approach or develop new methodologies for the modeling of the ramp to/from freeway maneuver. Some of the results and conclusions from the studies of the AASHTO and other models are presented in the following section.

A model of freeway merging was developed based on driver behavior in *Driver Behavior Model of Merging* by Michaels and Fazio [17]. In general, the model divided the merging process into initial ramp curve tracking and transition to the speed-change lane, a repetitive process of acceleration and gap search, and a final steering to the freeway lane or aborting the merge. A major deviation of this model from the AASHTO methodology is the concept of an iterative process between acceleration and gap search; that is, that the two events do not occur simultaneously and are performed in a repetitive process, one after the other. Based on the modeling of this behavior and the other aspects of the model, speed-change lane lengths were developed for ramp design speed vs. freeway lane volumes.

There are several notable conclusions that may be drawn from this study. The first involves the rate at which the recommended length of the speed-change lane decreases as the ramp design speed increases, compared to AASHTO. The AASHTO methodology leads to decreasing speed-change lane lengths as ramp design speed increases at a rate substantially greater than that of the Michaels and Fazio model. The irony of this study is that it leads the reader to the conclusion that the AASHTO guide may provide better operation at low ramp design speeds than at high ones. Although caution must be exercised in directly comparing this study with AASHTO (owing to limited sample size and potential differences in measuring speed-change length), the trend of reducing the speed-change lane length at a rate less than that utilized by AASHTO is clear. This conclusion is supported by the accident analysis in the previous section, where recommended minimum speed-change lane lengths are higher than those recommended by AASHTO for the higher ramp design speeds. A second point of interest in this study is that the required length of a speed-change lane decreases as the ramp volume increases. This point implies that when studying the operation of speed-change lanes, the critical operation for determining the acceptability of the design may actually occur during off-peak, lower volume periods.

A more detailed study with which the above study was connected is the unpublished NCHRP 3-35 *Speed-Change Lanes*, Final Report, 1989 [7]. This wide-ranging study evaluated current design guidelines for speed-change lanes, developed a model of speed-change lane operation, and developed new guidelines for speed-change lane design. Like the model in the previous study, this model attempted to capture the influence of traffic flow characteristics and driver behavior more accurately. The models developed attempted to integrate the human factor with geometry and vehicle operational characteristics. In addition to AASHTO requirements for speed-change lane design, this study also considered minimizing the disruption to freeway flow, meeting driver expectations, and avoiding overlapping control requirements for the driver. For example, this study defines ideal entrance ramp design as “one which minimizes the likelihood of overload and is adapted to the behavioral requirements of the entry process.” Furthermore, the design objective should be to “provide a static and dynamic environment that has maximum predictability for the driver.” For this study, design was based on the 85th driver percentile, meaning that 85 percent of the drivers should be able to complete the required maneuver (i.e., entry maneuver) in a shorter length than recommended.

Some differences between the assumptions of this study and the AASHTO study include that the AASHTO model bases speed-change lengths on operating speeds, which are lower than the design speeds, whereas NCHRP 3-35 assumes that operating speeds equal design speeds. In addition, the AASHTO 1990 guide defines the speed-change lane as beginning or ending at the 12 ft (3.7 m) taper; NCHRP 3-35 uses the 6 ft (1.8 m) point. Also, the NCHRP model utilizes several speeds along the length of the ramp and does not utilize the 5 mph (8 km/h) differential speed between the ramp vehicle and freeway operation as a merging threshold; instead, it utilizes an angular velocity threshold. These differences highlight the changing approach toward modeling speed-change lanes. When compared to the AASHTO speed-change lane length design values, the models developed in this study produced slightly shorter lengths at high freeway speeds and significantly longer lengths at moderate to low freeway speeds for acceleration lane lengths. It is also seen again that AASHTO may be too quickly reducing the speed-change lane lengths as speed differentials decrease.

These studies have clearly raised doubts about the applicability of AASHTO design and therefore about the TxDOT design standards based on AASHTO. It is possible not only that AASHTO’s acceptance of a minimum ramp design speed of 50 percent of the freeway design speed is inadequate, but also that the recommended AASHTO lengths may be too short at high speeds. Regarding the potential problem for TxDOT entrance ramp design standards, some of the higher ramp speed design concerns are alleviated since TxDOT utilizes a single design that provides greater than the AASHTO-recommended lengths for higher ramp design speeds, though the TxDOT design has been shown to possibly be shorter than that recommended in the 1990 guide for low ramp design speeds. With respect to deceleration, TxDOT's standard design faces the same issues as AASHTO, since the lengths it uses are simply a rounded version of the AASHTO lengths.

In discussion of the AASHTO acceleration and deceleration rates, several possible explanations of their applicability to today's design were mentioned. One possibility was that the 1938 acceleration and deceleration rates may not adequately compensate for deficiencies in the AASHTO model. These studies would lead to the conclusion that this is the likely situation.

SURVEY OF DESIGN PRACTICE

During the literature review, the researchers obtained an as-yet-unpublished survey of design agencies. Even though the survey does not deal explicitly with ramp design speed, the core ramp design speed issue is really that of speed change, and the survey deals directly with this element. Significant findings developed through the survey included the following: (1) Acceleration operations are viewed as more problematic than deceleration; (2) driver behavior during speed changes is not well characterized; (3) virtually all agencies rely on accident experience as the primary performance evaluation measure; (4) very little operational data describing speed change or ramp operations is collected; and (5) all agencies do not use the same design criteria. Additionally, effects of control devices, specifically ramp metering, are not well known.

Although this survey was conducted almost ten years ago, major changes in these findings are not very likely to occur. The literature review confirmed that little operational data have been collected either by agency activities or research efforts. The results of this survey tend to confirm concepts and problems discussed throughout this chapter.

STUDY AND REPORT OBJECTIVES

In this research effort, the study team traced freeway ramp design speed criteria contained in current AASHTO and TxDOT design policies through roughly 50 years of technical literature. In addition, the evolution of design speed criteria has been documented. This chapter showed that TxDOT ramp design speed criteria are, essentially, the AASHTO criteria. The origin of driver deceleration rates, which are built into AASHTO criteria, are the experimental studies performed during the late 1930s. Several studies have raised questions about the appropriateness of the AASHTO minimum allowable ramp design speed, which is 50 percent of the freeway design speed. Questions have also been raised about the adequacy of high-speed ramp lengths designed using the AASHTO criteria. Clearly, a thorough examination of current ramp design procedures is in order. Simply stated, this examination is the primary objective of this study and report. While this type of study may or may not produce recommendations for changing current ramp design speed criteria, the question regarding adequacy of current criteria will be answered in either case. Additionally, the analysis and primary data collected through the study will lead to a better understanding of freeway ramp operations. As noted earlier, freeway sections with ramps are usually primary freeway bottleneck locations and are, therefore, operationally critical. Improved understanding of operations in these critical freeway sections will constitute a secondary but significant benefit.

This study objective includes both a historical-theoretical and an experimental point of view, which leads to three implicit steps.

1. Evaluate, through literature review and analysis, current ramp design speed criteria.
2. Evaluate current ramp design speed criteria through a carefully designed sample of ramp operational data.
3. Provide sufficient evidence to validate current ramp design speed policy or recommendations modifying current procedures.

The first step has been taken in this chapter. The next two steps are carefully laid out and performed in the remainder of this report.

CHAPTER 2. FIELD DATA COLLECTION

As indicated in the previous chapter, evaluation of freeway entry ramp design speed criteria requires an examination of assumptions regarding ramp vehicle acceleration and deceleration rates, as well as of gap seeking and acceptance behavior. Such an examination should include freeway driver activity and ramp driver actions. This examination essentially involves recording the position and speed-time histories of entering and freeway main lane vehicles as they operate through freeway sections containing entry ramps.

Although a variety of measuring and recording techniques were considered, the study team determined that videotaping was the most practical technique. Since standard video cameras capture images at a rate of thirty frames per second, a video record would provide an adequate time resolution; that is, a vehicle traveling 60 mph (96 km/h) would travel less than 3 ft (10 m) during 1/30th second. However, video recording introduces a series of questions regarding camera angles, fields of view, distances from objects being viewed, and location and spacing of fiducial marks defining speed measurement resolution. Prior to selecting sites and beginning data collection, experiments and analyses were conducted to verify the appropriateness of video data collection.

VIDEO SPEED MEASUREMENT CAPABILITIES

An important advantage of video recording is that videotaping traffic provides a permanent record that can later be analyzed at various levels of detail or rechecked as necessary. The camera must be positioned in such a way that vehicle movements along the longitudinal direction can be clearly tracked. Considerable time and effort were expended in finding usable videotaping sites.

For the purpose of calculating speeds and acceleration-deceleration rates from video images, the acceleration lane was divided into specified distance intervals by painting (using completely biodegradable flour) lines on the ramp shoulder as fiducial marks. All lines were perpendicular to the pavement edge and were virtually invisible to drivers. The distance between lines was determined partially by distance from the video camera and partially by the required measurement accuracy.

In order to calculate vehicle speed and acceleration-deceleration rates, the times of crossing successive fiducial marks must be precisely read from the video image. Because not all available video cameras had built-in time code generators with satisfactory time-base resolution, a procedure that superimposed a crystal-controlled digital clock on completed recordings was developed. The time-base could be synchronized to any clock time before starting.

Data Reduction Procedure

Although the third chapter describes the detailed data reduction process, this section discusses several data reduction considerations that were important parts of the field data collection plan. For example, the speed and potential accuracy of data reduction would determine the quantity and nature of the experimental data. After examining automated and manual data reduction procedures, the study team procured image tracking software developed by CMS Engineering Systems of Long Beach, California, called "Mobilizer-PC." Although the software was able to track well-defined images quite reliably, camera movements, particularly vibrations, were problematic. Software tracking reliability was also dependent on lighting and on the angle between the camera axis and the earth's surface. A 90-degree angle produced best results, with any smaller angle impairing reliability. Also necessary was a lighting arrangement that produced significant contrast between vehicles and the pavement surface. Thus, the software application was constrained by limited camera angles, vibrations, and lighting problems. Consequently, most data reduction was performed manually.

Considerable effort was required to manually reduce videotape data. There were three primary tasks in the manual video data reduction. First, traffic counts were made by reviewing the videotapes in real time; second, individual vehicles were tracked along the merging area; and, finally, times and locations where ramp vehicles merge into the freeway were recorded. Locations of these occurrences were identified by the fiducial marks where they took place. Fiducial marks were extended across the acceleration lane and freeway lanes directly on a transparency superimposed on the video monitor.

The vehicle tracking process required videotapes to be played back at slow speed, or frame-by-frame, to ensure precise recording of the time vehicles crossed each fiducial mark. The video resolution permitted tracking vehicles at 0.03-second time intervals (30 frames/sec).

The primary data reduced from the videotapes were a set of times at which each ramp and right-lane freeway vehicle crossed each fiducial mark. The average speed of a vehicle between each pair of fiducial marks was calculated simply by dividing the distance between fiducial marks by the travel time. Acceleration and deceleration rates were calculated from the speed data. Also calculated was the longitudinal distance between a specific ramp vehicle and corresponding freeway lag, freeway lead, and ramp lead vehicles at the time when the ramp vehicle crossed each fiducial mark. This analysis procedure allowed the tracing of time-distance histories, or speed profiles, of ramp vehicles moving along the acceleration lane. Freeway vehicles were similarly traced.

Sources of Potential Measurement Errors

Data accuracy was an important research consideration. Accordingly, quality control was planned throughout the data collection and reduction process. Video measurement errors, especially those occurring in the vehicle tracking process, are significant. Difficulties in reducing data result from imbedded limitations of data collection devices, the visual blocking of vehicles by other vehicles, human error, or from the natural deficiencies of the adopted data reduction techniques.

Prior to the beginning of this study, an experiment was performed to examine potential reduced data consistency. Videotaping was conducted on Balcones Drive, a three-lane, one-way street. A video camera featuring 0.1 sec.(10 frames/sec) time-base resolution was used in this specific experiment. Two groups of vehicles, with twelve vehicles in each group, were sampled from video images. The times each vehicle passed two fiducial marks, 50 ft (15 m) apart, were recorded, respectively, and average travel speeds were calculated.

This process was repeated five times for each group. For each repetition, the same twelve vehicles were sampled. The reduced average travel speeds, in mph, of each vehicle in each repetition are shown in Tables 2.1 and 2.2, respectively. Essentially, if consistent data can be produced from video images, the average travel speeds for each vehicle in each repetition should be similar. In other words, the standard deviations shown in the last column of each table should be minor. Results in both tables show that most standard deviations are either zero or fairly small, indicating good potential consistency.

Another possible speed estimation measurement error results from video equipment limitations. In this study's video data collection effort, the video image was played back in a video camera recorder (VCR) that allowed video images to be moved forward/backward frame-by-frame. As a consequence, these calculated speeds have measurement errors caused by embedded video camera time-base resolution limitations. Ideally, if the time-base resolution is the only cause of measurement errors, the probability density functions (pdf) of this kind of measurement error, ε, associated with calculated average travel speed between fiducial marks is given in Equations (2.1) and (2.2) [25].

Table 2.1 Experimentally Derived Average Vehicle Travel Speeds in Each Repetition, Group 1

	Repetition						
Vehicle	1	2	3	4	5	**Mean**	**Std. Dev.**
1	24.30	24.30	26.16	24.30	24.30	24.67	0.83
2	28.34	28.34	28.34	28.34	28.34	28.34	0.00
3	28.34	28.34	28.34	28.34	28.34	28.34	0.00
4	28.34	28.34	28.34	28.34	28.34	28.34	0.00
5	30.92	30.92	30.92	30.92	30.92	30.92	0.00
6	28.34	28.34	28.34	28.34	28.34	28.34	0.00
7	26.16	26.16	26.16	26.16	26.16	26.16	0.00
8	28.34	26.16	26.16	26.16	28.34	27.03	1.19
9	28.34	28.34	28.34	28.34	28.34	28.34	0.00
10	28.34	28.34	26.16	28.34	28.34	27.90	0.97
11	24.30	24.30	24.30	24.30	24.30	24.30	0.00
12	28.34	28.34	28.34	26.16	28.34	27.90	0.97
Mean	27.70	27.52	27.49	27.34	27.70		
Std. Dev.	1.89	1.92	1.74	1.94	1.89		

Table 2.2 Experimentally Derived Average Vehicle Travel Speeds in Each Repetition, Group 2

	Repetition						
Vehicle	1	2	3	4	5	**Mean**	**Std. Dev.**
1	34.01	37.79	37.79	34.01	37.79	36.28	2.07
2	34.01	34.01	34.01	34.01	34.01	34.01	0.00
3	30.92	30.92	30.92	30.92	30.92	30.92	0.00
4	42.52	42.52	42.52	42.52	42.52	42.52	0.00
5	34.01	34.01	34.01	34.01	34.01	34.01	0.00
6	37.79	37.79	37.79	37.79	37.79	37.79	0.00
7	37.79	37.79	37.79	37.79	37.79	37.79	0.00
8	37.79	34.01	37.79	37.79	37.79	37.03	1.69
9	34.01	34.01	34.01	34.01	34.01	34.01	0.00
10	34.01	34.01	34.01	34.01	34.01	34.01	0.00
11	34.01	34.01	34.01	34.01	34.01	34.01	0.00
12	37.79	37.79	37.79	37.79	37.79	37.79	0.00
Mean	35.72	35.72	36.04	35.72	36.04		
Std. Dev.	3.07	3.07	3.08	3.07	3.08		

$$f(\varepsilon) = [\ \kappa - \frac{\kappa^2}{v_{act}} (\ \frac{\varepsilon D}{v_{act} + \varepsilon}\ )\ ] \frac{D}{(\ v_{act} + \varepsilon\ )^2} \tag{2.1}$$

$$0 \leq \varepsilon \leq \frac{v_{act}^2}{\kappa D - v_{act}}$$

$$f(\varepsilon) = [\ \kappa + \frac{\kappa^2}{v_{act}} (\ \frac{\varepsilon D}{v_{act} + \varepsilon}\ )\ ] \frac{D}{(\ v_{act} + \varepsilon\ )^2} \tag{2.2}$$

$$\frac{-v_{act}^2}{\kappa D + v_{act}} \leq \varepsilon \leq 0$$

where

κ = video camera time-base resolution, in frame/sec,

v_{act} = actual speed of a vehicle approaching the fiducial mark, in ft/sec, and

D = distance between each fiducial mark, in ft [25].

As expected, the faster the actual speed and the shorter the fiducial mark intervals, the larger the probability of having a large measurement error in estimating average travel speed between fiducial marks. Measurement error, which results from the time-base resolution, is not significant for a 30 frames/second resolution. The probability density function of measurement error can be derived from either time-scale or distance-scale orientation. The results from these two approaches have been proven to be identical. In addition, a Monte-Carlo simulation technique was applied to verify the pdf's derived mathematically. The goodness-of-fit chi-square test shows very good agreement.

However, the time-base resolution is not the only cause of measurement error. In tracking a vehicle, each frame is projected on a video display terminal with a perspective grid overlay. Parallax error inevitably occurs in determining when vehicles actually cross fiducial marks. The further down the acceleration lane the vehicle proceeds, for a given perspective distortion, the greater the errors are likely to be. The time-location errors propagate in the calculation of speeds, accelerations, and angular velocities. The parallax error is difficult to remove unless the fiducial marks can be painted directly on the pavement.

Travel Time Experiment

An experiment was designed to investigate the effects of fiducial mark distance on estimated travel speed consistency. The distance between each pair of fiducial marks was 10 ft (3.05 m) and the total marked area was 50 ft (15.25 m). Three experienced drivers were instructed to maintain constant speeds of 30, 40, 50, and 60 mph (48, 64, 80, 96 km/h), respectively, when passing the marked area. Each driver performed ten to fifteen runs for each driving speed. The experiment for 50 mph (80 km/h) and 60 mph (96 km/h) was performed in a freeway section with three through lanes, while the experiment for 30 mph (48 km/h) and 40 mph (64 km/h) was performed on a frontage road. All experiments were performed during weekend off-peak times to ensure that drivers received minimum disturbance from other vehicles and could easily maintain constant speed. The video time code resolution was 0.1 sec/frame. Data were reduced for fiducial mark distances 30, 40, and 50 ft (9.1, 12.2, and 15.2 m), respectively; results are shown in Table 2.3 [25].

Table 2.3 Results of Travel Speed Experiments

Test Car Speed	**Mean and Standard Deviation of Estimated Speed (mph) Fiducial Mark Distance (ft)**		
(mph)	30	40	50
30	28.58 (3.13)	27.26 (2.50)	27.26 (2.30)
40	35.75 (5.80)	35.66 (4.44)	36.06 (4.22)
50	48.27 (10.22)	50.93 (8.23)	52.66 (7.70)
60	54.22 (10.29)	54.32 (7.93)	54.56 (7.76)

Note: The value in parenthesis is the standard deviation

Although the standard deviations of test car speeds of 50 mph and 60 mph are almost the same, the trends are consistent, indicating that the larger the approach speed and the smaller the fiducial mark distance, the larger the estimated speed variance. However, increasing the fiducial mark interval reduces the speed measurement resolution; therefore, the fiducial mark interval choice must be a compromise between measurement error and measurement resolution. Application of this reasoning to actual choices for field measurements produced intervals of 30 to 60 ft.

DATA COLLECTION SITES

As indicated in Research Report 1732-1, a conceptual experiment design was developed during the first study year. This design was implemented to the greatest extent possible considering time, economic, and site constraints (e.g., observation position availability). The study team selected sites having "good" geometrics (e.g., sight distances, grades, and speed-change lane lengths) as well as sites having poor geometric features. Twenty sites were selected along freeways in Houston, San Antonio, Dallas, and Austin, Texas. Table 2.4 describes the sites and includes notes regarding where the video camera was located during recording. Note that in both Houston and San Antonio, significant data quantities were obtained using TranStar and TransGuide surveillance cameras. Personnel at both traffic control centers were exceptionally helpful and generous in providing access to their resources and valuable advice.

Table 2.4 Description of Field Observation Sites

Serial No.	City	Highway Name	Location Description	Camera Placement
1	Houston	IH 610 southbound, entrance from San Felipe Rd.	An urban area 5 miles west of downtown Houston	Marriott, Park Tower (South)
2	Houston	IH 45 northbound, lower level, entrance from Cullen Blvd.	An urban area 6 miles south of downtown Houston	TRANSTAR
3	Houston	IH 45 southbound, entrance between Victoria and Airline Dr.	An urban area approximately 7 miles north of downtown Houston	TRANSTAR

Serial No.	City	Highway Name	Location Description	Camera Placement
4	Houston	IH 45 southbound, entrance from W. Mount Houston Rd.	An urban area approximately 9 miles north of downtown Houston	TRANSTAR
5	Houston	IH 610 northbound, entrance from N. Braeswood Blvd.	An urban area 10 miles southwest of downtown Houston	TRANSTAR
6	Houston	IH 610 northbound, entrance from Richmond Ave.	An urban area 7 miles west/southwest of downtown Houston	TRANSCO Building
7	Houston	IH 610 northbound, entrance from San Felipe Rd.	An urban area 5 miles west of downtown Houston	Marriott, Park Tower (South)
8	San Antonio	IH 10 eastbound, lower level, entrance from Woodlawn Ave.	An urban area 1 1/2 miles northwest of downtown San Antonio	Cincinnati St. Bridge
9	San Antonio	HW 281 northbound, entrance from Isom Rd.	An urban area approximately 4 miles north of downtown San Antonio. The International Airport is 3/4 of a mile northwest of the ramp.	Roof of Office Building
10	San Antonio	IH 35 southbound, lower level, entrance from S. Alamo St.	An urban area 1/2 mile southwest of downtown San Antonio	TRANSGUIDE
11	San Antonio	HW 281 northbound, entrance from Josephine St.	An urban area 3/4 of a mile north of downtown San Antonio	TRANSGUIDE
12	Dallas	IH 75 northbound, entrance from Haskell Rd.	An urban area approximately 3 miles north of downtown Dallas	Roof of Office Building
13	Dallas	IH 75 northbound, entrance from Park Ln. (over Walnut Hill Ln.)	Four miles north of the Haskell ramp. An urban area. It is close to SMU and University Park, which is a prominent housing district.	Roof of Office Building
14	Dallas	IH 35 E. northbound, entrance from Mockingbird Ln.	Six miles northwest of the downtown area. An urban area with many business and office buildings nearby.	Roof of Office Building
15	Dallas	IH 35 E. southbound, entrance from Mockingbird Ln.	Six miles northwest of the downtown area. An urban area with many business and office buildings nearby.	Roof of Office Building
16	Dallas	IH 35 E. southbound, lower level, entrance from Empire Central Rd.	Approximately 1 mile north of the ramp at Mockingbird	Roof of Office Building
17	Austin	IH 35 southbound, lower level, entrance from Airport Blvd.	An urban area 4 miles north of downtown Austin. The Mueller Municipal Airport is 1/2 mile west of the ramp.	Bridge at 38 1/2 St.
18	Austin	IH 35 southbound, lower level, entrance from 38 1/2 St.	Approximately 3/4 of a mile south of the Airport Blvd. ramp	Bridge at 32 1/2 St.

Serial No.	City	Highway Name	Location Description	Camera Placement
19	Austin	IH 35 southbound, lower level, entrance from 32 1/2 St.	Six blocks south of the 38 1/2 St. ramp. The UT campus and the Capitol are approximately 1 1/2 miles south/southwest of this ramp.	Bridge at 38 1/2 St.
20	Austin	IH 35 northbound, entrance from Oltorf St.	An urban area approximately 3 miles south of downtown Austin.	Roof of Office Building

Table 2.5 presents additional information, including approximate speed-change lane lengths and other notable details. Acceleration lane lengths are measured from the painted ramp gore or nose until a full-width lane is no longer available as a result of taper or lane termination.

Table 2.5 Field Site Geometric Characteristics

No.	City	Highway Name	Speed Change Lane Length (approximate ft)	Condition Details
1	Houston	IH 610 southbound, entrance from San Felipe Rd.	600	No protective barrier at end of taper lane
2	Houston	IH 45 northbound, lower level, entrance from Cullen Blvd.	640	No protective barrier at end of taper lane
3	Houston	IH 45 southbound, entrance between Victoria & Airline Dr.		Temporary entrance ramp in construction zone, no taper lane, concrete barriers on shoulder
4	Houston	IH 45 southbound, entrance from W. Mount Houston Rd.	420	No protective barrier at end of taper lane
5	Houston	IH 610 northbound, entrance from N. Braeswood Blvd.	430	Two highways merge where ramp meets highway, no protective barrier at end of taper lane
6	Houston	IH 610 northbound, entrance from Richmond Ave.	540	No protective barrier at end of taper lane
7	Houston	IH 610 northbound, entrance from San Felipe Rd.	630	Highway is on a curve where entrance ramp joins, no protective barrier at end of taper lane

No.	City	Highway Name	Speed Change Lane Length (approximate ft)	Condition Details
8	San Antonio	IH 10 eastbound, lower level, entrance from Woodlawn Ave.	400	Limited visibility for drivers before entering to highway, narrow shoulder, concrete barrier on shoulder
9	San Antonio	HW 281 northbound, entrance from Isom Rd.	360	No protective barrier at end of taper lane
10	San Antonio	IH 35 southbound, lower level, entrance from S. Alamo St.	600	Limited visibility, concrete barrier, narrow shoulder
11	San Antonio	HW 281 northbound, entrance from Josephine St.	460	Sharp curve on highway, no protective barrier at end of taper lane
12	Dallas	IH 75 northbound, entrance from Haskell Rd.	180	Construction zone, no right shoulder, concrete barrier on right shoulder
13	Dallas	IH 75 northbound, entrance from Park Ln. (over Walnut Hill Ln.)	350	Limited visibility by concrete barriers, concrete barrier on right shoulder, highway is on higher level than ramp and ramp inclines upward
14	Dallas	IH 35 E. northbound, entrance from Mockingbird Ln.	340	
15	Dallas	IH 35 E. southbound, entrance from Mockingbird Ln.	370	No protective barrier at end of taper lane
16	Dallas	IH 35 E. southbound, lower level, entrance from Empire Central Rd.		Highway slope declines while ramp is on incline
17	Austin	IH 35 southbound, lower level, entrance from Airport Blvd.	430	Limited visibility by concrete barriers, concrete wall causes drivers to feel uncomfortable/ unsafe, driver perceives as dangerous
18	Austin	IH 35 southbound, lower level, entrance from 38 1/2 St.	790	Limited visibility by concrete barriers, concrete wall causes drivers to feel uncomfortable/ unsafe, driver perceives as dangerous
19	Austin	IH 35 southbound, lower level, entrance from 32 1/2 St.	505	Limited visibility by concrete barriers, concrete wall causes drivers to feel uncomfortable/unsafe, driver perceives as dangerous

No.	City	Highway Name	Speed Change Lane Length (approximate ft)	Condition Details
20	Austin	IH 35 northbound, entrance from Oltorf St.	335	Limited visibility, highway slope is on decline while ramp is on incline

As indicated in Table 2.6, most video data collection was performed during the spring and summer of 1997; however, several Austin area sites were observed during the summer of 1998 and one data set was taken from a previous 1995 research effort. Estimates of hourly traffic volume were developed for every site in order to guide choices of videotaping times. Most sites were congested during at least one peak period; thus, during these times, freeway and ramp vehicle speeds were suppressed. Video data collection times were specially chosen to represent typical conditions, so congestion caused by incidents was either avoided or included but specifically noted. Slightly less than 200 hours of video data are identified in Table 2.6.

Table 2.6 Field Data Collection Dates and Times

Serial No.	City/Highway	Date (s)	Times
1	Houston IH 610 S. and San Felipe Rd.	5/17/95 Wednesday	8:30 A.M. – 9:40 A.M. 10:10 A.M. – 10:55 A.M. 6:25 P.M. – 7:55 P.M.
2	Houston IH 45 N. and Cullen Blvd.	6/11/97 Wednesday	11:18 A.M. – 1:20 P.M. 4:48 P.M. – 6:50 P.M.
3	Houston IH 45 S. b/w Victoria and Airline Dr.	6/11/97 Wednesday 6/12/97 Thursday	9:02 A.M. – 11:02 A.M. 11:54 A.M. – 1:57 P.M. 4:50 P.M. – 6:52 P.M. 10:36 A.M. – 10:57 A.M.
4	Houston IH 45 S. and W. Mt. Houston Rd.	6/12/97 Thursday	8:15 A.M. – 9:09 A.M. 9:10 A.M. – 10:16 A.M. 11:02 A.M. – 12:00 P.M. 4:09 P.M. – 6:12 P.M.
5	Houston IH 610 N. and N. Braeswood Blvd.	6/12/97 Thursday	8:22 A.M. 10:25 A.M. 11:01 A.M. – 1:03 P.M. 4:15 P.M. – 6:17 P.M.

Serial No.	City/Highway	Date (s)	Times
6	Houston IH 610 N. and Richmond Ave.	7/09/97 Wednesday 7/10/97 Thursday 7/20/97 Sunday 7/24/97 Thursday	3:19 P.M. – 5:00 P.M. 10:53 A.M. – 12:55 P.M. 12:56 P.M. – 2:56 P.M. 2:58 P.M. – 4:29 P.M. 9:47 A.M. – 11:47 A.M. 11:49 A.M. – 1:49 P.M. 1:52 P.M. – 3:52 P.M. 11:30 A.M. – 1:30 P.M. 1:33 P.M. – 3:33 P.M.
7	Houston IH 610 N. and San Felipe Rd.	7/09/97 Wednesday 7/10/97 Thursday 7/20/97 Sunday 7/24/97 Thursday	3:58 P.M. – 5:00 P.M. 11:40 A.M. – 1:40 P.M. 1:48 P.M. – 3:48 P.M. 9:10 A.M. – 11:10 A.M. 11:26 A.M. – 1:26 P.M. 1:31 P.M. – 3:31 P.M. 10:45 A.M. – 12:45 P.M. 1:00 P.M. – 3:00 P.M.
8	San Antonio IH 10 E. and Woodlawn Ave.	8/07/97 Thursday	10:44 A.M. – 12:44 P.M. 12:50 P.M. – 2:50 P.M. 2:55 P.M. – 4:55 P.M.
9	San Antonio HW 281 N. and Isom Rd.	8/07/97 Thursday	9:25 A.M. – 11:25 A.M. 11:29 A.M. – 1:29 P.M. 2:00 P.M. – 4:00 P.M.
10	San Antonio IH 35 S. and S. Alamo St.	8/06/97 Wednesday 8/07/97 Thursday	12:15 P.M. – 2:15 P.M. 4:30 P.M. – 6:30 P.M. 9:43 A.M. – 11:43 A.M. 12:00 P.M. – 2:00 P.M. 2:30 P.M – 4:30 P.M.
11	San Antonio HW 281 N. and Josephine St.	8/06/97 Wednesday 8/07/97 Thursday	12:15 P.M. – 2:15 P.M. 4:30 P.M. – 5:41 P.M. 9:43 A.M. – 11:43 A.M. 12:00 P.M. – 2:00 P.M. 3:00 P.M. – 4:32 P.M.
12	Dallas IH 75 N. and Haskell Rd.	8/26/97 Tuesday	11:45 A.M. – 1:41 P.M. 1:45 P.M. – 3:44 P.M. 3:45 P.M. – 5:06 P.M.
13	Dallas IH 75 N. and Park Ln.	8/25/97 Monday	3:25 P.M. – 5:27 P.M. 11:17 A.M. – 1:18 P.M. 1:20 P.M. – 3:21 P.M. 2:24 P.M. – 5:19 P.M.
14	Dallas IH 35 N. and Mockingbird	8/25/97 Monday	2:30 P.M. – 4:32 P.M.
15	Dallas IH 35 S. and Mockingbird	8/26/97 Tuesday	10:00 A.M. – 12:03 P.M. 12:37 P.M. – 2:40 P.M. 2:45 P.M. – 4:48 P.M.

Serial No.	City/Highway	Date (s)	Times
16	Dallas IH 35 S. and Empire Central Rd.	8/25/97 Monday 8/26/97 Tuesday	2:30 P.M. – 4:32 P.M. 9:47 A.M. – 11:49 A.M. 12:35 P.M. – 2:37 P.M. 2:45 P.M. – 4:48 P.M.
17	Austin IH 35 S. and Airport Blvd.	6/23/98 Tuesday 6/25/98 Thursday	12:10 P.M. – 2:10 P.M. 2:20 P.M. – 3:20 P.M. 3:25 P.M. – 4:25 P.M. 9:20 A.M. – 10:20 A.M. 10:25 A.M. – 11: 25 A.M.
18	Austin IH 35 S. and 38 1/2 St.	7/16/98 Thursday	10:35 A.M. – 12:33 P.M. 12:40 P.M. – 2:38 P.M. 2:45 P.M. – 4:46 P.M.
19	Austin IH 35 S. and 32 1/2 St.	7/30/98 Thursday	10:30 A.M. – 12:30 P.M. 12:35 P.M. – 2:35 P.M. 2:35 P.M. – 4:34 P.M.
20	Austin IH 35 S. and Oltorf	8/11/98 Tuesday 8/12/98 Wednesday	11:45 A.M. – 1:43 P.M. 2:00 P.M. – 4:03 P.M. 9:20 A.M. – 11:21 A.M. 11:25 A.M. – 1:26 P.M.

SUMMARY

Video data collection was implemented at twenty field sites in Houston, San Antonio, Dallas, and in Austin, Texas. Geometric features at selected locations ranged broadly from excellent to poor. The next chapter describes the video data reduction procedures.

CHAPTER 3. DATA REDUCTION PROCESS AND ANALYSIS PROCEDURE

Chapter 2 described the field data collection effort. This chapter concentrates on the data reduction and analysis procedure applied to the collected field data. This procedure involves a very detailed vehicle time-position data collection effort, an effort that underlies the calculated operational characteristics and subsequent statistical analysis. Chapter 4 will then present a discussion of the analysis results.

OVERVIEW OF DATA REDUCTION PROCESS AND ANALYSIS PROCEDURE

Figure 3.1 is a flow chart of the data reduction and analysis procedure. The steps preceding this process are the ramp selection and field data collection discussed in Chapter 2. A synopsis of each step is presented below. The remainder of this chapter is an in-depth discussion of each step.

Volume Counts. Volume counts are completed in five-minute increments for each two-hour field data video tape. These five minutes volumes are then converted to hourly flow rates.

Select Analysis Time. Based on the hourly flow rates determined in step 1, and on the sample size requirements, time periods are chosen for in-depth analysis.

Record Fiducial Mark Crossing Times. Time-position data is collected for every vehicle on the right most freeway and ramp lanes during the selected analysis times. Data collected includes, the hour, minute, second, and frame that each vehicle crosses each mark and ramp vehicle merge and highway vehicle lane-change locations.

Create Computer Files. The time-position data is transferred into a spreadsheet format and converted from hour, minutes, seconds, and frames into a more usable seconds format. Also, single correlated data sets are created for ramps with multiple data sets resulting from multiple camera locations.

Determine Performance Characteristics. Next, the speed, acceleration, deceleration, headway (freeway vehicles only), accepted gap (ramp vehicles only) and merge position is determined for each vehicle.

Statistical Analysis. A statistical analysis including mean, standard deviation, fifteenth percentile and eighty-fifth percentile is computed for the vehicle performance characteristics for each analysis time period.

Presentation of Analysis. Graphical and tabular representations of the statistical analysis are created which allow for the study of the operating characteristics along a ramp during an individual time period, along a ramp during different time periods and between different ramps.

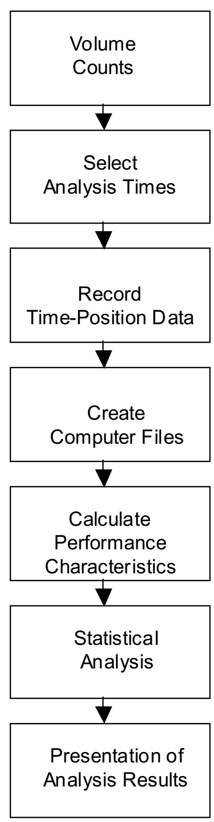


Figure 3.1 Data Reduction Process and Analysis Procedure

Volume Counts

As outlined in Chapter 2, the research team typically utilized 2-hour videotaping time periods. An analysis of each 2-hour period in its entirety would require an insurmountable effort, as several man-hours are often required to collect seconds of real-time vehicle time-position data. The reduction of this data collection effort to a manageable size, while maintaining desired stable volume levels, required selecting 10- to 20-minute segments from the 2 hours of potential data for an in-depth analysis. In selecting this shorter period, it is necessary to have flow rate data for the entire 2-hour period. Thus, the first task in the data reduction effort was to conduct vehicle volume counts.

Volume count data are collected manually through videotape observation within a laboratory setting. Fortunately, volume counting can be performed at nearly real-time, which is not possible with later vehicle time-position data collection. The basic procedure involves observing a ramp video tape in real-time while counting vehicles in one to several lanes. Thus, with one or two runs through a video, volumes are collected for all ramp and freeway lanes. These volumes are recorded in five and fifteen minute time increments. Finally, these volumes are converted into hourly flow rates to be used in the selection of time periods for in-depth data collection.

Selection of Analysis Times

The ability to study only portions of each 2-hour time raises two important questions: (1) when to begin the collection of data for the shorter time period, and (2) what length time period should be used. The answer to these questions relies on the calculated flow rates and the desired statistical precision of the overall performance characteristics.

The answer to the first question, when to collect data, is based on the flow rates determined through the volume counting effort. Time periods are selected to represent different flow rate combinations on the freeway and ramp lanes. For example, during one 15-minute period the right-lane freeway and ramp flow rates could be 1,500 vphpl and 800 vphpl, respectively, while during a different 15-minute period they could be 1,000 vphpl and 600 vphpl. Thus, both of these periods could be selected as analysis periods, since they capture differing flow characteristics. If another time period is seen to have the same or nearly the same flow characteristics, it is eliminated from detailed analysis.

The second question regarding what length time period should be used relates directly to the desired accuracy of the operational characteristics statistical measures. Intuitively, it is clear that the longer the time period studied, the more data points collected, and the higher the likelihood of accurate statistics; that is, one is clearly more confident in an average speed based on 100 vehicles than one based on 10. Unfortunately, the trade-off presented is that, as confidence increases, so does the required data collection effort. A balance may be achieved through an application of the central limit theorem:

$$n = (z\,\sigma / h)^2 \tag{3.1}$$

where

n = minimum required sample size,

σ = population standard deviation,

z = parameter based on standard normal distribution, and

h = desired measurement precision.

This relationship provides a quantitative description of the relationship between desired precision and sample size. For example, consider the question, How many vehicle speeds should be collected to achieve 95 percent confidence that the resulting mean is within 2 mph (3.2 km/h) of the actual population mean? First, it is necessary to decide the desired level of measurement precision, i.e., in the given example, it is desired that the sample mean speed be within 2 mph (3.2 km/h) of the population mean speed. There exist no fixed rules or guidelines for determining the required precision level; such a determination is a judgment based on experience and experimental requirements. Also required in this example is the population standard deviation of speed, which, unfortunately, is unknown and must be estimated from a pilot sample, although, fortunately, it can be shown that as the sample size increases the sample standard deviation approaches the population standard deviation. So, continuing with the speed example, a conservative standard deviation is estimated to be 10 mph (16 km/h) (based on previous studies). Thus, the required sample size would be $(10*1.96/2)^2$ or approximately 100 vehicles. Of course, as any analysis progressed the standard deviation for that particular data set could be utilized to refine this estimate and to determine if additional data are required for the conditions represented by the particular data set.

This required number of data points estimation may be repeated for each performance characteristic. Utilizing data from earlier experiments and from initial experimentation in this study, it was determined that a minimum of 75 to 100 data points on both the ramp and the highway could provide for reasonable accuracy while not presenting an insurmountable data collection requirement. When possible, superior accuracy is provided through the collection of additional data points.

Collection of Time-Position Data

Once the times to be analyzed have been selected, the process of collecting vehicle time-position data is initiated. As mentioned, this can be a time-consuming task, where several man-hours could equate to the collection of only a few seconds of real-time data.

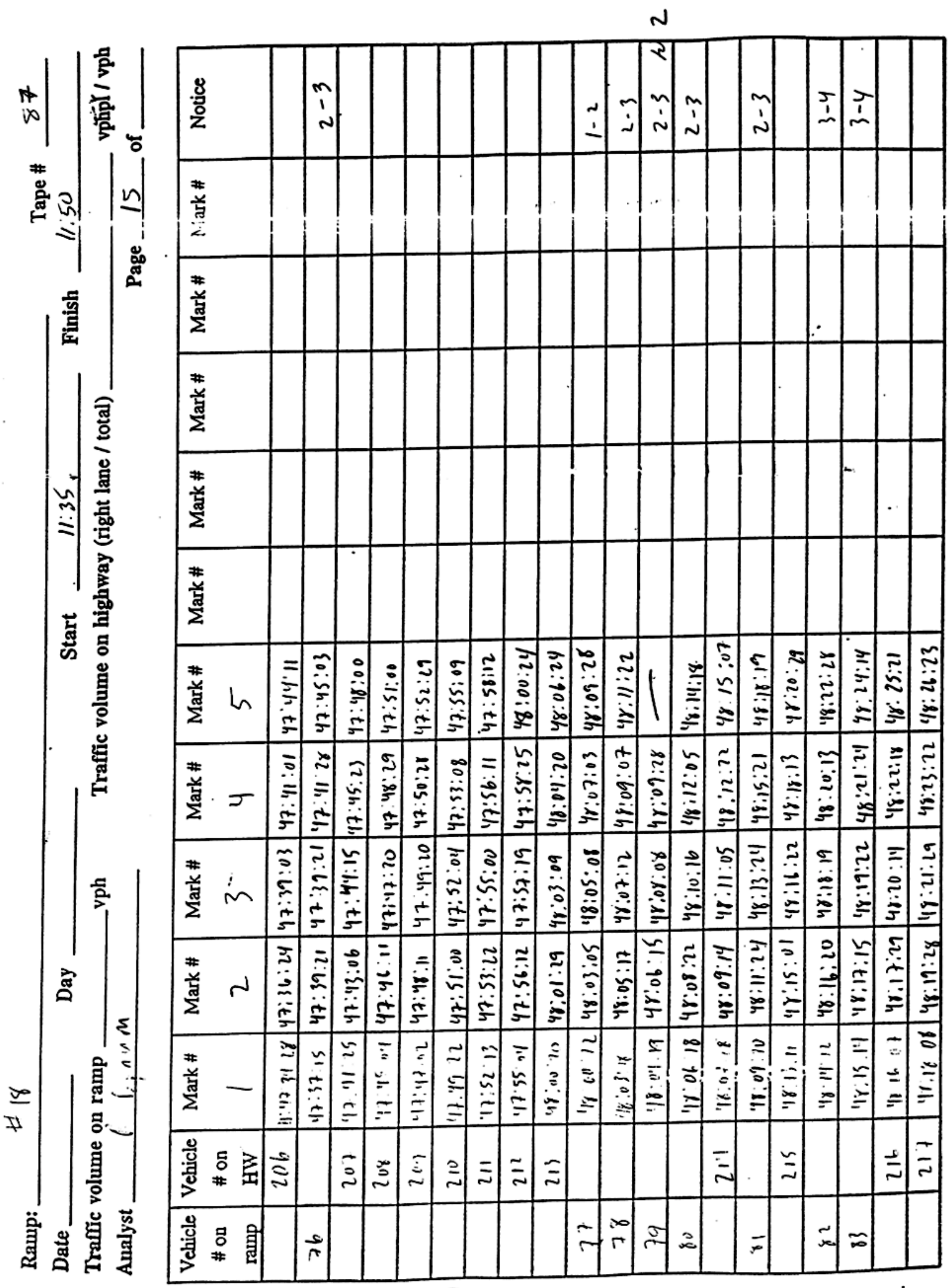
The first step in this process is the set up of the video player and monitor. The videotape is begun at the beginning of the analysis time period so as to permit display of the video picture on the monitor. Referencing the visible roadside fiducial marks sketches and photographs from the sight, the fiducial marks are extended across the ramp and freeway lanes onto a transparency superimposed on the video monitor. The video player and monitor unit is then dedicated to the data collection for that time period. In order to minimize potential errors that could result from multiple setups for one time period, video player and monitor units are not switched back and forth between different analysis periods or videotapes; this practice would also allow for maximum data consistency.

As mentioned previously, a time code generator is used to encode each videotape frame with an hour, minute, second, and frame number. One “frame” of videotape is 1/30th of a second, the smallest time increment registered by the video recorders available during this study. Thus, when a vehicle crosses a fiducial mark, its position is recorded to the nearest 30th of a second. This hour, minute, second, and frame data are recorded by an observer for all vehicles on the ramp and right-most freeway lanes. No data was collected for vehicles in other freeway lanes. To provide as much accuracy and consistency as possible, each observer is instructed to chose a point on each vehicle (such as the front bumper or tire) and record the fiducial mark crossing time as the time at which that reference point is closest to the fiducial mark. To aid in the data collection and later analysis each vehicle and fiducial mark is assigned a unique number that is recorded along with the fiducial mark crossing times.

The observer also records where a ramp vehicle merges and if and where a right-lane freeway vehicle changes lanes. As only the positions of the fiducial marks are accurately known, it is not possible to record the exact location of a merge maneuver. Only the marks between which the merge occurred may be noted. Also, as fiducial marks range from 60 to 120 feet (18 to 37m) apart, it is possible for a vehicle to use more than this distance in the completion of a merge maneuver. Therefore, to maintain consistent and reliable merge location data, the observers are instructed to record the fiducial marks just upstream and downstream of the point at which the merging vehicle first breaks the plane of the destination lane.

Data Formatting

Once the time-position data are collected for a time period, the next data reduction step process is to transfer the handwritten data into a spreadsheet. Recorded in the spreadsheet for each vehicle is its unique identification number and the minute, second, and frame at which the vehicle crossed each fiducial mark. Also, the marks at which the vehicle conducted any merge maneuvers are entered into the spreadsheet. Figures 3.2 and 3.3 are example raw data and spreadsheet data pages.



Ramp: #18 Tape # 87
Date ______ Day ______ Start 11:35 Finish 11:50
Traffic volume on ramp ______ vph Traffic volume on highway (right lane / total) ______ vphpl / vph
Analyst [illegible] Page 15 of ______

Vehicle # on ramp	Vehicle # on HW	Mark # 1	Mark # 2	Mark # 3	Mark # 4	Mark # 5	Mark #	Mark #	Mark #	Mark #	Mark #	Notice
	206	11:47:34:28	47:36:24	47:39:03	47:41:01	47:44:11						
76		47:37:15	47:39:21	47:39:21	47:41:28	47:45:03						2-3
	207	47:41:25	47:43:06	47:44:15	47:45:23	47:48:00						
	208	47:45:01	47:46:11	47:47:20	47:48:29	47:51:00						
	209	47:47:02	47:48:11	47:49:20	47:50:28	47:52:29						
	210	47:49:22	47:51:00	47:52:04	47:53:08	47:55:09						
	211	47:52:13	47:53:22	47:55:00	47:56:11	47:58:12						
	212	47:55:01	47:56:12	47:57:19	47:58:25	48:00:24						
	213	48:00:20	48:01:29	48:03:09	48:04:20	48:06:24						
77		48:00:12	48:03:05	48:05:08	48:07:03	48:09:28						1-2
78		48:03:18	48:05:17	48:07:12	48:09:07	48:11:22						2-3
79		48:04:19	48:06:15	48:08:08	48:09:28	—						2-3 2
80		48:06:18	48:08:22	48:10:16	48:12:05	48:14:18						2-3
	214	48:07:18	48:09:14	48:11:05	48:12:22	48:15:07						
81		48:09:20	48:11:24	48:13:24	48:15:21	48:18:19						2-3
	215	48:13:11	48:15:01	48:16:22	48:18:13	48:20:29						
82		48:14:12	48:16:20	48:18:19	48:20:13	48:22:28						3-4
83		48:15:11	48:17:15	48:19:22	48:21:24	48:24:14						3-4
	216	48:16:07	48:17:29	48:20:11	48:22:18	48:25:21						
	217	48:18:08	48:19:28	48:21:29	48:23:22	48:26:23						

Figure 3.2 Raw Data Spreadsheet

Mean								
	Mark 1	**Mark 2**	**Mark 3**	**Mark 4**	**Mark 5**	**Mark 6**	**Mark 7**	**Mark 8**
1	3.63	3.61	3.64	3.87	3.90	2.90	2.51	2.33
2	3.64	3.64	3.62	3.84	3.73	2.91	2.43	2.17
3	3.46	3.43	3.40	3.64	3.61	2.94	2.53	2.06
4	3.25	3.21	3.24	3.42	3.40	2.65	2.21	2.03
5	2.97	2.96	3.00	3.22	3.23	2.73	2.28	1.84

Standard Deviation								
	Mark 1	**Mark 2**	**Mark 3**	**Mark 4**	**Mark 5**	**Mark 6**	**Mark 7**	**Mark 8**
1	3.65	3.67	3.69	4.03	4.17	2.49	2.18	2.07
2	3.87	3.89	3.58	3.63	3.40	2.48	1.98	1.79
3	3.01	3.00	2.98	3.20	2.98	2.37	1.90	1.58
4	3.06	3.04	3.06	3.55	3.56	2.36	1.79	1.70
5	2.40	2.41	2.41	2.69	2.69	2.03	1.77	1.36

85%								
	Mark 1	**Mark 2**	**Mark 3**	**Mark 4**	**Mark 5**	**Mark 6**	**Mark 7**	**Mark 8**
1	6.63	6.67	6.74	6.76	6.82	5.09	4.20	3.98
2	5.82	5.86	6.02	6.59	6.38	5.17	3.83	**3.51**
3	5.96	5.75	5.84	6.66	6.87	5.16	4.28	3.34
4	5.66	5.57	5.47	5.73	5.71	4.43	3.51	3.13
5	4.97	5.04	5.07	5.66	5.67	4.90	3.75	3.07

Median								
	Mark 1	**Mark 2**	**Mark 3**	**Mark 4**	**Mark 5**	**Mark 6**	**Mark 7**	**Mark 8**
1	2.23	2.20	2.23	2.37	2.37	1.97	1.77	1.60
2	2.32	2.25	2.33	2.57	2.690	2.03	1.77	1.58
3	2.38	2.33	2.27	2.40	2.53	2.17	1.97	1.53
4	2.13	2.13	2.10	2.10	2.10	1.80	1.63	1.52
5	2.13	2.07	2.10	2.30	2.45	1.93	1.67	1.33

15%								
	Mark 1	**Mark 2**	**Mark 3**	**Mark 4**	**Mark 5**	**Mark 6**	**Mark 7**	**Mark 8**
1	1.09	1.10	1.10	1.07	1.08	1.03	0.97	0.90
2	1.13	1.13	1.11	1.10	1.07	1.03	0.97	0.89
3	1.10	1.12	1.10	1.20	1.17	1.10	0.97	0.87
4	1.07	1.10	1.11	1.10	1.17	0.97	0.87	0.80
5	1.01	1.00	1.03	1.00	1.03	1.00	0.87	0.77

Figure 3.3 Headway Statistics, Entrance Ramp, IH 35 N. Oltorf Street, Austin, Texas

A later goal of this data reduction and analysis process is the development of speed, acceleration, headway, accepted gap, and merge point profiles for each vehicle. Before these profiles may be developed, it is necessary to convert the raw data into a more usable format. This involves converting the minute-second-frame data into overall second values. This is simply a matter of multiplying minutes by 60, adding seconds, and adding frames divided by 30. This conversion provides the time in seconds at which each vehicle crosses each fiducial mark. Table 3.1 provides an example of five hypothetical vehicles time-position data for a ramp with six fiducial marks. Table 3.2 is a conversion of the data into total seconds.

Table 3.1 Time-Position: Minute, Seconds, Frames

Ramp Vehicle Number	Right lane Vehicle Number	Mark 1			Mark 2			Mark 3			Mark 4			Mark 5			Mark 6			R to 1	1 to 2	2 to 1
		mm	ss	fr	mm	ss	fr	mm	ss	fr	mm	ss	fr	mm	ss	fr	mm	ss	fr			
	56	20	4	3	20	6	5	20	7	22	20	9	7	20	10	20	20	12	4			
	57	20	23	25	20	25	21	20	27	6	20	28	22	20	30	9	20	31	24			
21		20	27	8	20	29	8	20	30	24	20	32	9	20	33	22	20	35	4	3		
22		20	54	21	20	56	18	20	58	5	20	59	21	21	1	5	21	2	17	5		
	58	20	57	1	20	59	11	21	1	4	21	2	26	21	4	15	21	6	7			

mm - minutes, ss - seconds, fr - frame
R to 1 - mark number after which ramp vehicle begins merge
1 to 2 - mark number after which vehicle begins to merge from lane 1 to lane 2
2 to 1 - mark number after which vehicle begins to merge from lane 2 to lane 1

Table 3.2 Data in Seconds

Ramp Vehicle Number	Right lane Vehicle Number	Mark 1	Mark 2	Mark 3	Mark 4	Mark 5	Mark 6	R to 1	1 to 2	2 to 1
	56	1204.10	1206.17	1207.73	1209.23	1210.67	1212.13			
	57	1223.83	1225.70	1227.20	1228.73	1230.30	1231.80			
21		1227.27	1229.27	1230.80	1232.30	1233.73	1235.13	3		
22		1254.70	1256.60	1258.17	1259.70	1261.17	1262.57	5		
	58	1257.03	1259.37	1261.13	1262.87	1264.50	1266.23			

Several ramps also required one additional data formatting step. Owing to ramp configurations and availability of camera-mounting locations, some ramps were divided into two or more sections for observation. This division could result in as many as three cameras recording a ramp during an analysis period. Thus, before any analysis could be performed, it was necessary to combine multiple data sets resulting from multiple cameras. This task requires correlating the vehicle numbers from the different tapes. This is accomplished through a combination of visually tracking vehicles from one tape to another and through several computer procedures developed specifically for this purpose.

Operational Characteristics

Five primary operational characteristics are calculated for ramp and right-lane vehicles: speed, acceleration, headway (freeway right lane vehicles only), accepted gap (ramp vehicles only), and merge point (ramp vehicles only). As will be seen in subsequent chapters, these five characteristics provide a telling picture of the ramp, right lane, and merging operation. While the usefulness of these measures will be seen later, this section's primary concern is to provide an accurate description of how each of these measures is calculated.

Figure 3.4 provides a sample sketch of a typical ramp layout. This sketch includes six fiducial marks at which vehicle time-position data would be collected. Tables 3.1 through 3.5 provide an example of the data reduction and analysis process for five hypothetical vehicles for this ramp.

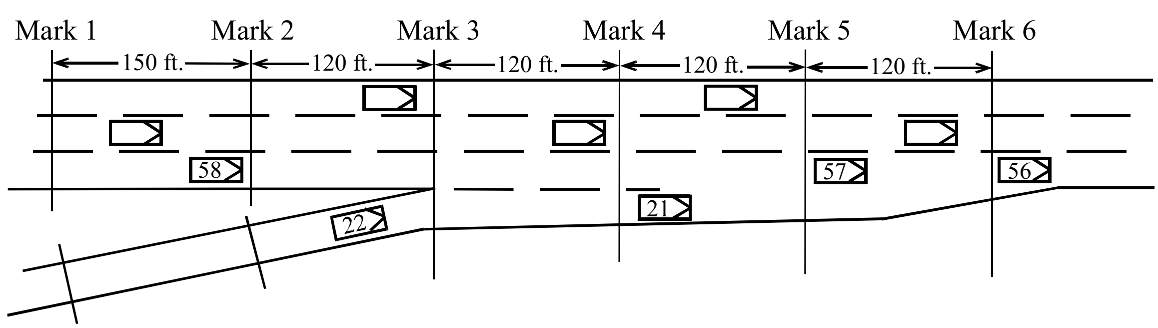


Figure 3.4 Headway Statistics, Entrance Ramp, IH 35 N. Oltorf Street, Austin, Texas

Speed Calculation. The first operational characteristic calculated from the time-position data is speed. The available data at this point in the analysis process is the time in seconds at which each vehicle crosses each fiducial mark and the distance in feet between the fiducial marks. With this information, the determination of the speed is a straightforward matter of calculating the travel time between two fiducial marks and then dividing the distance by this time. The calculated speed, in ft/s, is then converted into mph. For example, vehicle 57 in Table 3.2 crosses mark 2 at time t = 1225.70 sec and mark 3 at time t = 1227.20 sec. This equates to a travel time from mark 2 to mark 3 of (1227.20 - 1225.70) or 1.5 seconds. The distance between these marks is 120 ft (137 m) (Figure. 3.4). Utilizing this information, the speed from mark 2 to mark 3 is calculated as 120ft./1.5sec. (37m/1.5m), which is 80 ft/s (24m/s) or 54.55 mph (86.96 km/h).

The following question may be raised: Is the calculated speed constant between the fiducial marks and, if not, where exactly does this speed occur between the two fiducial marks? Unfortunately, the level of detail provided by the raw data does not allow for an absolute answer to this question. Therefore, a conservative approach is adopted in a subsequent analysis. When a point on the roadway must be identified as the speed location, it is taken to be midway between the two fiducial marks.

Table 3.3 Vehicle Speeds

Ramp Vehicle Number	Right lane Vehicle Number	Mark 1 to Mark 2	Mark 2 to Mark 3	Mark 3 to Mark 4	Mark 4 to Mark 5	Mark 5 to Mark 6
	56	49.49	52.22	54.55	57.08	55.79
	57	54.79	54.55	53.36	52.22	54.55
21		51.14	53.36	54.55	57.08	58.44
22		53.83	52.22	53.36	55.79	58.44
	58	43.83	46.31	47.20	50.09	47.20

Acceleration Calculation. Acceleration is the next operational characteristic to be calculated. Acceleration is simply a measure of the rate of change of speed — in other words, the change in speed per unit time. To calculate acceleration as stated it is necessary to know a vehicle's speed change over some measure of time. Unfortunately, the information available for this analysis is a vehicle's speed change over some measure of distance. Fortunately, acceleration may still be calculated by using the less intuitive Equation (3.2).

$$a= ((V^2 - Vo^2)/(2(X-X_o)))/1.47 \qquad (3.2)$$

For example, vehicle 58 was determined to have speeds of 43.83 mph (70.58 km/h) and 46.31 mph (74.57 km/h) from mark 1 to mark 2 and mark 2 to mark 3, respectively. As discussed previously, these are taken to be spot speeds centered between the given fiducial marks. Therefore, knowing the distance from mark 1 to mark 2 and from mark 2 to mark 3 to be 150 ft (45.7 m) and 120 ft (36.6 m), respectively, the distance between the spot speed measurements is calculated as (150/2 + 120/2), which equals 135 ft (41.1 m). Utilizing Equation 3.2, with an initial speed of 43.83 mph (70.58 km/h), a final speed of 46.31 mph (74.57 km/h) and distance of 135 ft (41.1 m), an acceleration of 1.21 mphps (1.95 km/h) is obtained.

Table 3.4 Vehicle Accelerations

Ramp Vehicle Number	Right lane Vehicle Number	speed 1,2 / speed 2,3	speed 2,3 / speed 3,4	speed 3,4 / speed 4,5	speed 4,5 / speed 5,6
	56	1.51	1.51	1.73	-0.89
	57	-0.14	-0.78	-0.73	1.51
21		1.26	0.78	1.73	0.96
22		-0.92	0.73	1.62	1.85
	58	1.21	0.51	1.72	-1.72

Headway Calculation. There are two primary ways by which headways are typically measured, namely, in time or in distance. Time headway is defined as the time interval between the moment at which the front of one vehicle passes a point to the moment the front of the next vehicle passes the same point. Distance headway is defined as the distance between the front of one vehicle and the front of the following vehicle at any given moment in time. Owing to the nature of the data collected in this study, time headways are utilized. For each vehicle, at each mark, a time headway is calculated. The time headway for a vehicle is calculated as the time difference from when a vehicle crosses a fiducial mark and the nearest downstream vehicle crossed the same mark. For example, in Table 3.2 one may note that vehicle 56 crosses mark 5 at time t = 1210.67 seconds and vehicle 57 crosses mark 5 at time t = 1230.30 seconds. Therefore, the headway for vehicle 57 is 19.63 seconds, the difference between the crossing times of vehicles 56 and 57.

The headways are determined for all vehicles in the right freeway lane for this report, as this was hypothesized to be a potentially significant factor in ramp vehicle merging operations. The primary complication in this calculation is that which is introduced by merging ramp vehicles. Clearly, a vehicle in the ramp lane should not be considered in the right-lane headway calculations. However, having once merged, the ramp vehicle is part of the right-lane operations and is, therefore, considered within the right-lane headway calculations. The general rule used is to consider a merging ramp vehicle as part of right-lane operations as soon as the respective ramp vehicle breaks the plane of the right lane. Table 3.5 is an example of calculated vehicle headways. Right-lane vehicles have headways determined at every mark, whereas ramp vehicles have headways determined only after beginning their merge maneuver.

Table 3.5 Vehicle Headways

Ramp Vehicle Number	Right lane Vehicle Number	Mark 1	Mark 2	Mark 3	Mark 4	Mark 5	Mark 6	R to 1	1 to 2	2 to 1
	56	5.54	5.62	5.42	5.46	5.51	5.47			
	57	19.73	19.53	19.47	19.50	19.63	19.67			
21		NA	NA	NA	3.57	3.43	3.33	3		
22		NA	NA	NA	NA	NA	27.43	5		
	58	33.20	33.67	33.93	30.57	30.77	3.67			

Accepted Gap Calculations. The accepted gap of a ramp vehicle is the time headway of the two freeway vehicles between which the ramp vehicle merges. Figure 3.5 provides several examples of merging ramp vehicles and the associated freeway vehicles used in the accepted gap calculation.

For example, in Figure 3.5a, vehicle 20 is merging. Vehicles 10 and 11 are the associated downstream and upstream vehicles between which vehicle 20 will merge. Vehicle 10, the downstream vehicle, is considered the lead vehicle and vehicle 11, the upstream vehicle, is considered the lag vehicle. Thus, vehicle 11's headway is the same as vehicle 20's accepted gap. Furthermore, the time headway of vehicle 20 to vehicle 10 is considered the lead time and the time headway from vehicle 11 to vehicle 20 is considered the lag time.

It is, of course, possible to find merging situations more complicated than those found in Figure 3.5a. Several examples of more complicated merging situations are given in Figures 3.5b and 3.5c. When considering which vehicle acts as the lead vehicle, the guideline applied is that the lead vehicle is always the closest downstream vehicle, whether located in the freeway right lane or the ramp speed-change lane. For example, in Figure 3.5b there are several ramp vehicles preparing to merge. In this example, ramp vehicle 20's lead vehicle is 10 and ramp vehicle 21's lead vehicle is 20, i.e., the nearest downstream vehicle. In the determination of the lag vehicle, the guideline applied is that the lag vehicle is the nearest upstream vehicle in the freeway right lane. Therefore, when two ramp vehicles are entering the freeway, the upstream ramp vehicle is the lag vehicle of the downstream only if it merges to the right lane first. For example, in Figure 3.5b the lag vehicle of both ramp vehicles 20 and 21 is upstream vehicle 11, while in Figure 3.5c vehicle 21 is the lag vehicle of vehicle 20, as vehicle 21 has begun its merge maneuver earlier than vehicle 20. Vehicle 11 is still the lag vehicle associated with vehicle 21's merge maneuver.

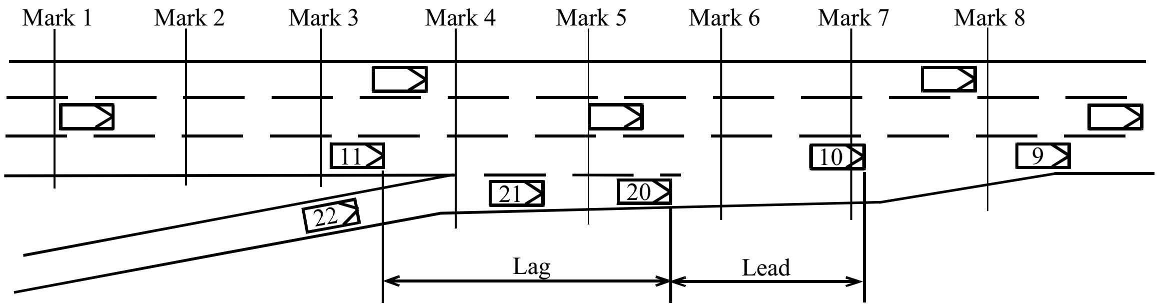


Figure 3.5a Merge Point Determination, Example 1

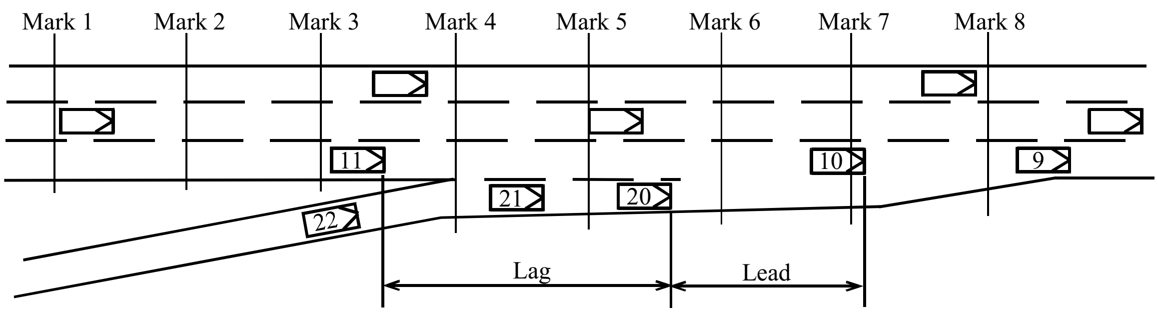


Figure 3.5b Merge Point Determination, Example 2

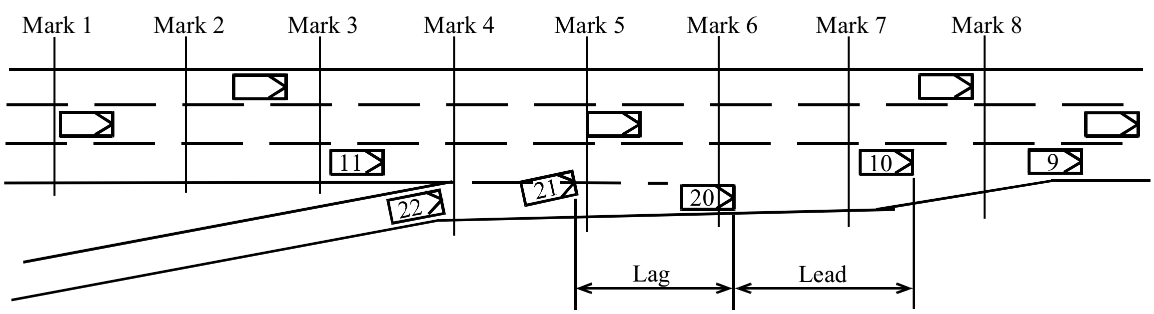


Figure 3.5c Merge Point Determination, Example 3

INDIVIDUAL VEHICLE ANALYSIS AND ERROR CHECKING PROCESS

With the vast quantities of data collected, it was important to develop an analysis procedure that allowed for consistency, clarity, and accuracy. Figure 3.6 is a representation of the procedure by which the analysis was conducted for each ramp data set once the vehicle time-position data collection is complete. The steps in this procedure are discussed below.

Initial Error Checking. This involved a visual inspection of the data sets. Errors captured by this checking procedure were obviously incorrect second values, missing seconds data, missing merging data, missing-lane change data, and incorrect and missing vehicle numbers. Any errors found during this initial check are immediately corrected.

Error Check. This step involves performing an error check analysis of the speeds and accelerations determined. This analysis is conducted two parts. The first part is a visual inspection of the speeds and accelerations. Readily observable errors are identified through this inspection. These errors may include such items as excessively high acceleration or decelerations, unusually high or low speeds, negative speeds, or missing data. In addition to the visual inspection, an automated inspection is also performed. This inspection identifies speeds and accelerations outside of predetermined limits. Any data points highlighted through either the visual or automated inspection are checked for potential errors. Potential error sources include transcribing mistakes, incorrect vehicle correlations in multicamera situations, video monitor set-up errors, and erroneous fiducial mark crossing times.

Complete Analysis. Once the speed and acceleration analysis is error checked, a complete analysis is performed. In addition to determining speeds and accelerations, this analysis determines headways, accepted gaps, and merge points.

Final Error Check. The final step in this stage of the analysis is a final error checking. This involves a visual inspection of all performed analysis. If any errors are uncovered, corrections are made and the analysis is repeated until no errors are realized.

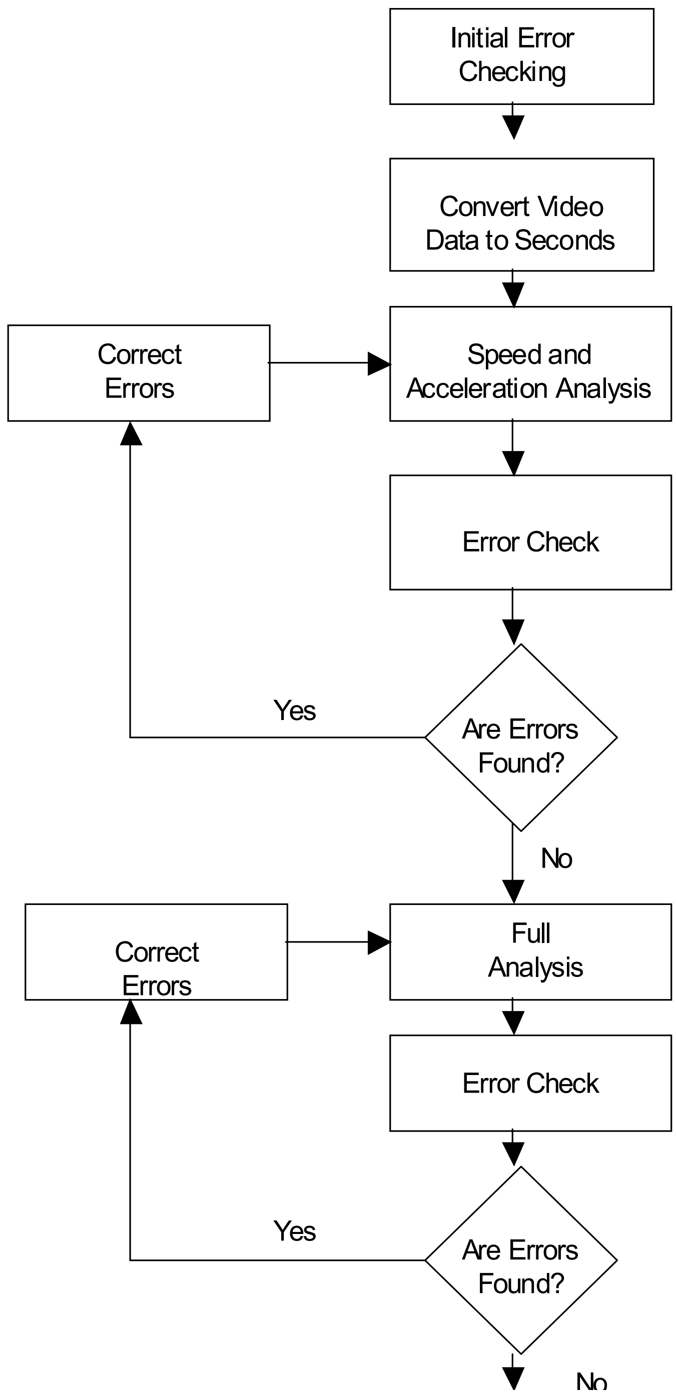


Figure 3.6 Speed, Acceleration, Headway, Accepted Gap and Merge Point Analysis

Statistical Analysis

Once the operational characteristics are determined for each vehicle throughout an analysis period, summary statistics are calculated. The statistics utilized are mean, standard deviation, 85th percentile, and 15th percentile. These measures were chosen because, when taken together, they provide a concise, simple, and understandable description of the ramp and freeway operations. While additional potential statistical measures were also considered, these were eliminated as they were deemed to offer little additional insight into the roadway operations.

The statistical measures are determined for each operational characteristics data point along the ramp or highway lanes. For example, headways are determined at each mark along the freeway. Therefore, for each mark a mean, 85th percentile, 15th percentile, and standard deviation of headway is determined. Later in this report it will be seen that this level of statistical analysis allows one to compare the operation between different ramps, the operation of an individual ramp during different time periods, and the operational changes along an individual ramp during a particular time period. The following briefly discusses each of these statistical measures.

Mean. The mean is simply the arithmetic average. The means given in this report are clearly sample means, which provide estimates of population means. The means were calculated according to the following formula;

$$\bar{x} = \frac{\sum_{i=1}^{n} x_i}{n} \tag{3.3}$$

where

n = number of observations, and
x_i = value of observation *i*.

Standard Deviation. The standard deviation provides a measure of the variability of the operational characteristics under study. For example, a ramp where most vehicles operate at the same general speed will have a low speed standard deviation, whereas a ramp where vehicles operate over a wide range of speeds will have a higher speed standard deviation. The standard deviations were calculated according to the following formula:

$$s_x = \sqrt{\frac{\sum_{i=1}^{n}\left(x_i - \bar{x}\right)^2}{n-1}} \tag{3.4}$$

85th Percentile. The 85th percentile provides useful insight into the upper range of a particular operating characteristic. Simply stated, this statistic provides the value below which 85 percent of vehicles operate for a given operating characteristic.

15th Percentile. The 15th percentile provides useful insight into the lower range of a particular operating characteristic. Simply stated, this statistic provides the value below which 15 percent of vehicles operate for a given operating characteristic.

SUMMARY

This chapter has described the field data reduction and analysis procedures. Chapter 4 presents findings developed through implementation of these procedures.

CHAPTER 4. SPEED-DISTANCE HISTORY ANALYSES

Previous chapters have described field studies of freeway entry ramp operations. The resulting speed-distance histories of ramp and freeway vehicles contain massive amounts of significant information. Graphical representations of those data were developed to specifically address this study's fundamental research question: Should the current entry ramp design speed criteria be modified? Answering this question requires an investigation not only of the relationships between entry ramp geometric design features and speed, but of other operational characteristics as well. One means of studying such relationships was comparing operational characteristics of ramps having "good" geometrics versus those having poor geometrics. Generally, ramps characterized as having "good" geometrics are those exceeding the AASHTO and TxDOT criteria, while those characterized as poor only marginally meet or fail to meet current criteria.

INTRODUCTION TO OBSERVATIONS

The following discussions offer operational characteristic comparisons of six different freeway entry ramps. The operating characteristics compared are speed, acceleration/deceleration, headway (freeway right-lane vehicles only), merging location (ramp vehicles only), and accepted gap (ramp vehicles only). Operating characteristics are considered for each ramp in its entirety and in two sections, upstream and downstream of the ramp gore. The fact that some operating characteristics behave differently in these two regions has important design implications.

Observation Types

Three primary types of observations may be made for each operating characteristic. The first comparison relates to observed characteristic changes occurring along the ramp under particular volume conditions. For example, a study area may have 1,000 and 500 vehicles per hour on the freeway right-lane and ramp, respectively. For this condition, speed observations are made as one traverses the ramp facility, with such observations meant to identify how speed changes (or does not change) as vehicles travel downstream. This type of comparison will typically be referred to as "along the ramp."

The second comparison type consists in comparing operational characteristics of an individual ramp under different volume combinations (i.e., ramp volume and right-lane volume pairs). Data have been collected for three to six volume combinations for each of the six ramps. In this type of comparison, the focus will typically be on apparent trends, the absence of any predictability, or similarities between volume combinations. This type of comparison is typically referred to as "between volume combinations."

The third comparison type compares different ramps. Observations are made regarding operations on different ramps under similar volume conditions. Important observations include differences in operating characteristics variability and operating characteristic magnitudes.

Also, two different descriptive terms will be utilized. The first is "smooth," which is intended to describe a ramp where only minimal operating characteristic changes occur as one travels from data point to data point. Figure 4.1 is an excellent example of "smooth" speed operations. The speed varies little from speed measurement to speed measurement, but an overall trend such as decreasing speed may exist.

The second term that will be utilized is "waveform." An operating characteristic that exhibits a waveform poses more radical differences between consecutive data points, i.e., it graphically has the appearance of a "wave." Figure 4.2 is an excellent example of this type of waveform behavior.

Ramp Data Utilized

The primary focus of observations within this chapter will be on the entrance ramps from Richmond Avenue to NB IH 610 (Houston) and from Oltorf Street to NB IH 35 (Austin). Respectively, these ramps provide excellent examples of "good" and "bad" design features. The Richmond Avenue entry ramp and adjacent freeway main lanes (1) have essentially no grade; (2) ramp drivers can see freeway traffic from the frontage road and all ramp elements; and (3) the speed change lane length exceeds current criteria.

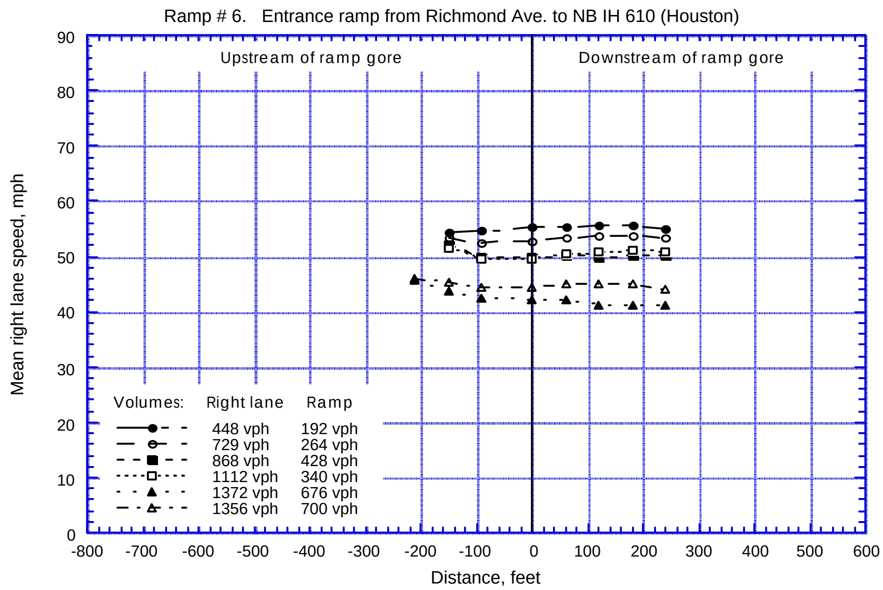


Figure 4.1 Mean Freeway Right Lane Speed, Richmond Avenue to Northbound IH 610, Houston, Texas

On the other hand, the Oltorf entry ramp and adjacent freeway lanes have grades that limit a ramp driver's view of freeway traffic until they pass the ramp gore. Also, the speed-change lane length does not meet current length criteria.

Observations based on the comparisons of these two ramps will then be further supported through additional entrance ramp observations, including San Felipe Road to NB IH 610 (Houston), considered an acceptably designed ramp; 38th Street to SB IH 35 (Austin) and Haskell Road to NB US 75 (Dallas), both considered marginal designs; and, finally, Airport Boulevard to SB IH 35 (Austin), considered a substandard ramp design.

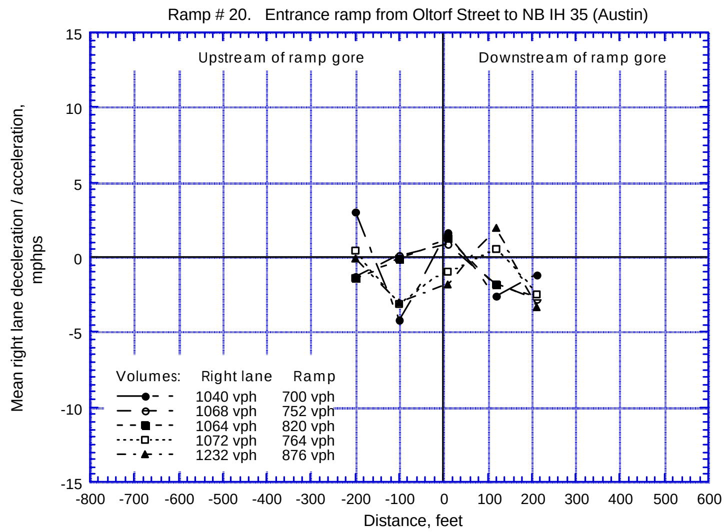


Figure 4.2 Mean Freeway Right Lane Acceleration/Deceleration, Oltorf Street to northbound IH 35, Austin, Texas

RAMP CHARACTERISTICS OBSERVATION DISCUSSION

Ramp Speeds *(Figures 05–08, Appendices A–F)*

Ramp speed is one of the more telling ramp characteristics. Graphics depicting mean, standard deviation, 85th percentile, and 15th percentile ramp speeds for six example entry ramps are presented in Appendices A through F as Figures 05 through 08. For the Richmond Avenue, ramp speed is seen to be smooth for all studied volume combinations. As indicated in Figure 4.3, several volume combinations exhibit a slight increasing trend, while the other volume combinations exhibit unchanging speeds.

The behavior seen on the Oltorf Street ramp is strikingly different. For all volume combinations the speed is seen to "dip" in the gore area. As shown in Figure 4.4, mean speeds expressed in miles per hour tend toward the high 40s both upstream and downstream of the gore and to the mid- to lower-40s in the gore area.

Another operational difference is seen in the greater variation in mean speeds occurring between consecutive measurement points on the Oltorf Street ramp as compared with the Richmond Avenue ramp. This variation is witnessed further in the peaking of the speed standard deviation in the gore area on Oltorf Street compared to the relatively constant (and in general somewhat lower) standard deviation along the entire length of the Richmond Avenue ramp (Figures 4.5 and 4.6). While these trends and observations are all based on the mean ramp speed, they are also observed to hold with the 85th and 15th percentile speeds.

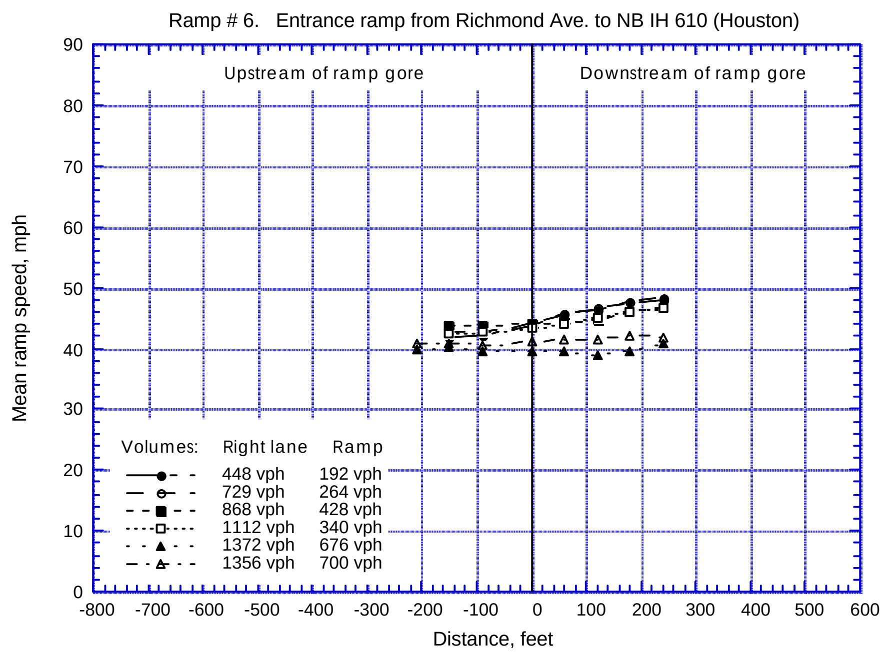


Figure 4.3 Mean Ramp Speed Versus Distance, Richmond Avenue to NB IH 610, Houston, Texas

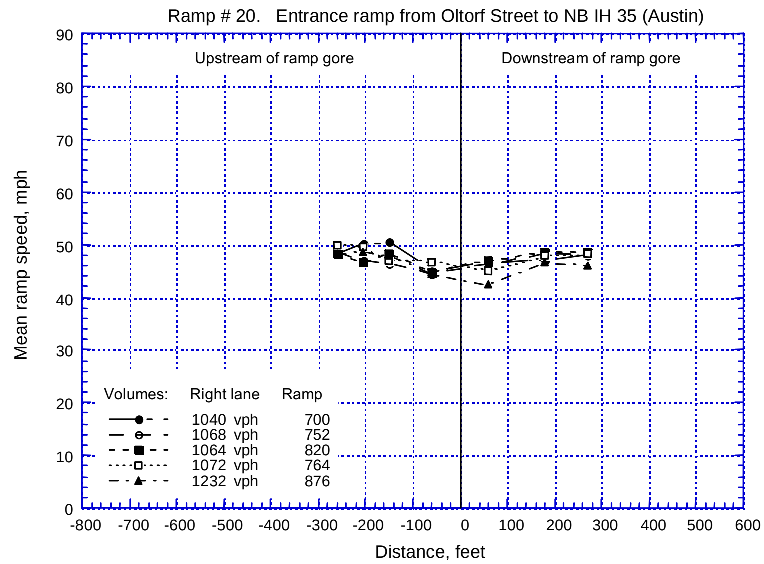


Figure 4.4 Mean Ramp Speed Versus Distance, Oltorf Street to NB IH 35, Austin, Texas

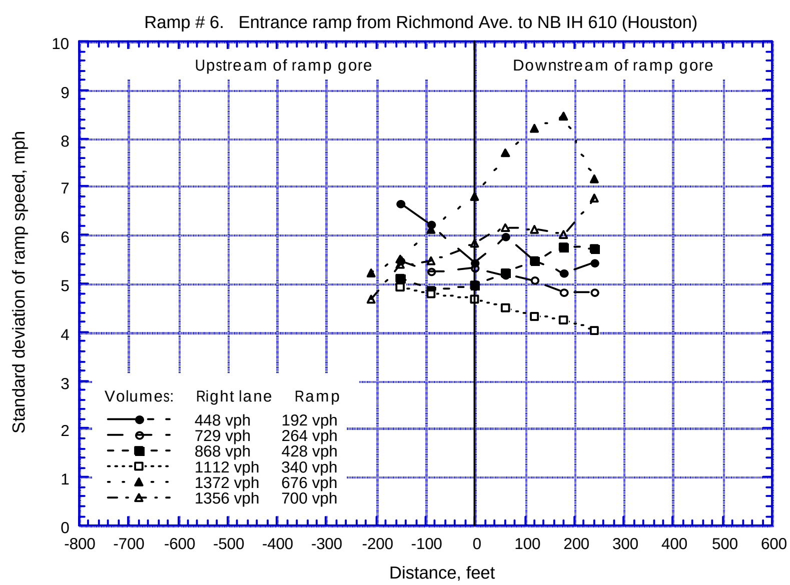


Figure 4.5 Standard Deviation of Ramp Speed Versus Distance, Richmond Avenue to NB IH 610, Houston, Texas

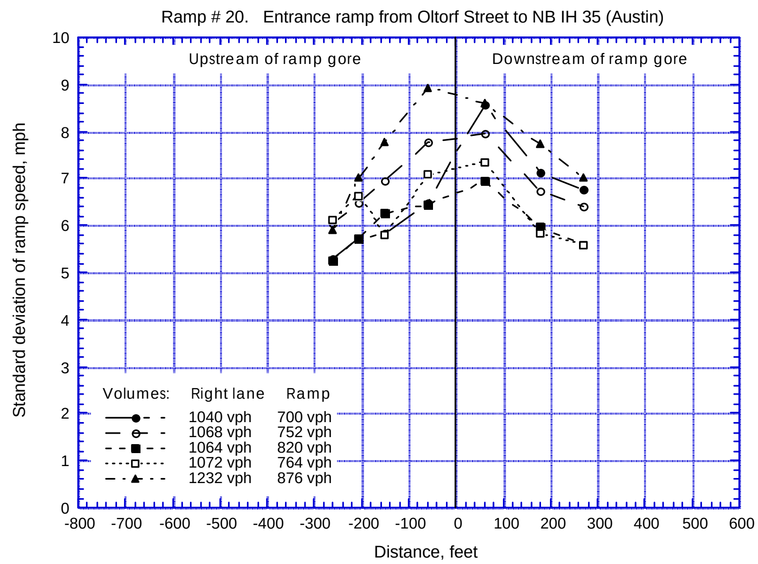


Figure 4.6 Standard Deviation of Ramp Speed Versus Distance, Oltorf Street, Austin, Texas

These observations suggest several hypotheses that will be further supported in the study of the other ramps and other operating characteristics. The first hypothesis is that a low design standard has a negative effect on ramp operations upstream of the gore. Of specific importance is sight-distance from the ramp to the upstream freeway lanes. For example, due to vertical alignment constraints this sight-distance is practically nonexistent prior to the Oltorf Street ramp gore. Drivers are not able to begin their gap search process or any meaningful analysis of the freeway operation until they are in the ramp gore area. At this point, having an imminent need to merge, drivers must direct primary attention to the freeway traffic, only peripherally watching for ramp vehicles that might occupy their current path. This situation prompts deceleration, or at least inhibits acceleration. On the Richmond Avenue ramp the driver's view of the freeway traffic is not limited by any vertical alignment feature. This unrestricted view allows the driver to task share between navigating the ramp and preparing for the freeway merge. Thus, at no point along the "good" ramp does the typical driver feel compelled to focus complete attention on the freeway traffic.

This stated hypothesis is supported by the observations of the other ramps. The 38th Street and the Airport Boulevard ramps both have extremely limited sight distances prior to the ramp gore. Both exhibit low speeds in the gore area followed by notable increases downstream. In addition, both tend to have notably higher speed standard deviations in the gore area. The complete "dip" phenomena — deceleration when the freeway traffic first becomes visible — is not replicated on these ramps, as adequate upstream data are not available. However, both ramps connect to frontage roads where operating speeds are typically greater than the first measured ramp speed, indicating that drivers apparently do decelerate when entering the ramp facility. The San Felipe Road ramp, a "good" ramp, displays the speed characteristics of the Richmond Avenue ramp. It exhibits relatively smooth operation for all volume combinations, an increasing speed trend from upstream to downstream, and relatively consistent speed standard deviations along the ramp length.

The most compelling support for the suggested hypothesis derives from the Haskell Road ramp observations. This ramp is considered a "marginal" design, primarily a result of its having a short downstream speed-change length (prompted by construction conditions). The upstream section of the ramp does have reasonable sight distance to the freeway lanes. Speed characteristics are seen to exhibit the trend of the "good" ramps: vehicle speeds increase as one travels downstream. There is no evidence of any type of speed-dipping phenomenon, though, as a result of the shorter distance over which speed changing must occur, speeds increase at a rate faster than those observed for the Richmond Avenue and San Felipe Road ramps.

Along with the given comparisons, the research team noted some general information concerning the observed ramp speed characteristics and their relation to design. Of probably the greatest interest is the 85th percentile speed. On the "good," "bad," and marginal ramps, 85th percentile speeds in the 40s and 50s (mph) are observed, both upstream and downstream of the ramp gore. This observation lends support to the position that, regardless of a ramp's design, drivers in the 85th percentile region will attempt to drive at speeds within 70 to 80 percent of the typical 70 mph freeway design speed. Eighty-fifth percentile speeds in the 35 mph range (50 percent of 70 mph) were not observed on any of the ramps. Furthermore, on San Felipe Road, on Richmond Avenue, and on Oltorf Street, mean upstream ramp speeds are in the 40+ mph range. Not until the 15th percentile speed level are speeds in the mid- and low 30s (mph) consistently observed on the ramps.

Ramp Accelerations and Decelerations *(Figures 13–16, Appendices A–F)*

A review of ramp-lane acceleration and deceleration characteristics offers additional insight into the effects of design on vehicle operations. As shown in Figure 4.7, mean acceleration/deceleration rates on the Richmond Avenue ramp follow a smooth, low value (maximum roughly 2 mphps) for all ramp volume combinations.

Several volume combinations exhibit mean acceleration/deceleration values that hover around 0. The maximum change in acceleration/deceleration rate measured between two consecutive points along the ramp for any volume combination does not exceed roughly 1 mphps and, except for the lowest ramp volume scenario, the standard deviations maintain a smooth, consistent centering around approximately 1 mphps. As indicated in Figure 4.8, the Oltorf Street ramp displays different characteristics. Upstream of the ramp gore the acceleration/deceleration rates are highly variable, both from location to location along the ramp and at the same location under different volume combinations. There is no observable or predictable trend except for the wide scattering of possible acceleration/deceleration values, with each volume combination following a different waveform.

Compared with the Richmond Avenue's maximum difference of approximately 1 mphps between two consecutive points, the Oltorf Street ramp experiences a wide range of differences, up to roughly 7 mphps. The Oltorf Street ramp also exhibits an upstream standard deviation under the different volume conditions that is consistently higher than that of Richmond Avenue. The variation and magnitude seen downstream of the gore on the Oltorf Street ramp is not as pronounced as that seen upstream, but it still exceeds that of the Richmond Avenue ramp, both along the ramp and between volume combinations.

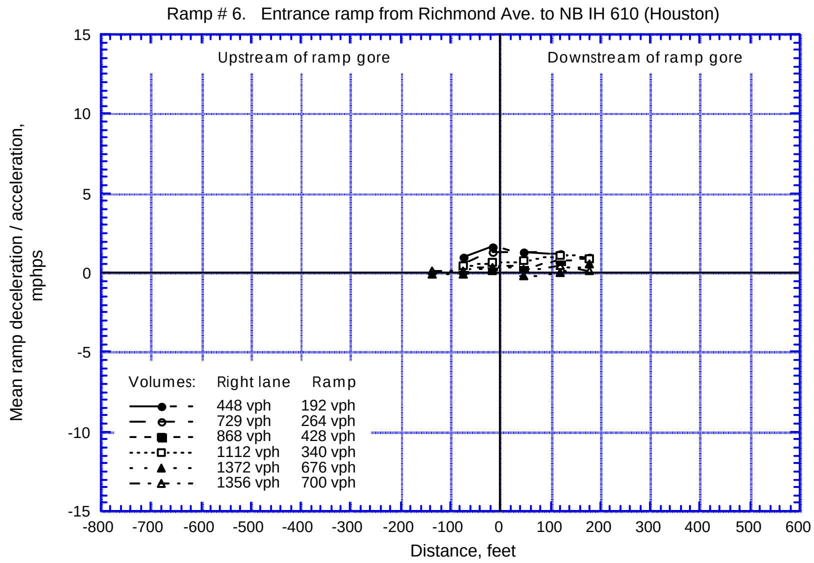


Figure 4.7 Mean Ramp Acceleration/Deceleration, Richmond Avenue to NB IH 610, Houston, Texas

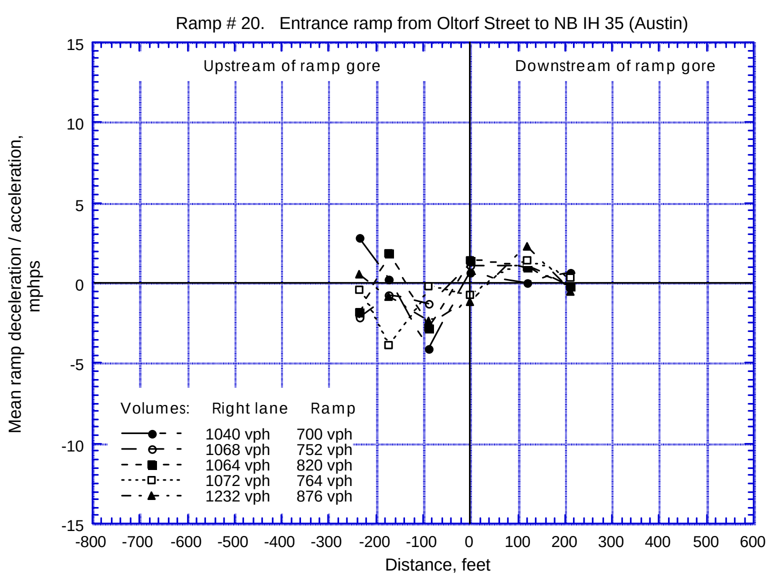


Figure 4.8 Mean Ramp Acceleration/Deceleration, Oltorf Street to NB IH 35, Austin, Texas

The discussed acceleration/deceleration observations further support the hypothesis that insufficient upstream design negatively impacts vehicle operation. In addition, the acceleration/deceleration rates reflect the effects of short speed-change lengths. It is observed that, on "good" ramps, smaller, incremental speed adjustments may be made, as opposed to the sharper speed changes required on the poorer ramp designs. These sharper changes are due both to the lack of upstream sight distance, which eliminates valuable merge preparation time, and to shorter downstream lengths, which force more rapid speed changes. When provided adequate upstream design, the drivers are able to gauge the freeway operation at a much earlier point in time and space, allowing for earlier and less dramatic adjustments in preparation for merging. In addition, a compounding of the effects of inadequate upstream sight distance and short speed-change length may contribute to the highly variable upstream operation. As upstream drivers are unable to observe the freeway lanes, they find themselves in the position of trying to catch their first freeway glimpse while responding to vehicle operations on the downstream ramp section and while attempting to anticipate appropriate merging clearances. This is a high-demand situation in which the driver is supplied inadequate information, leading to overly dramatic vehicle actions and to a propagation of downstream acceleration/deceleration back to the upstream area. This situation may be described as one in which an upstream driver acquires a general sense of "driving blind" with respect to merging.

The trends upon which these hypotheses are based are also seen on the other ramps studied. As expected, the San Felipe Road ramp follows the same smooth consistency along the ramp and shows volume combinations similar to those of the Richmond Avenue ramp. Additionally, the standard deviations behave similarly, with only minor changes in values along the ramp length. The 38th Street ramp, which is considered a moderate design, exhibits acceleration/deceleration rates that are higher than those of the well-designed Richmond Avenue ramp, but not the dramatic waveforms seen in the poorly designed Oltorf Street ramp. The Haskell Road ramp shows more consistent trends in acceleration/ deceleration among volume combinations, an observation reflective of adequate upstream sight distance; the ramp also shows a waveform trend with accel/decel rates higher than those of the "good" ramps, reflecting the short downstream distance over which speed changes must be conducted.

Freeway Right-Lane Speed *(Figures 1–4, Appendices A–F)*

In addition to the insight gained through study of the effects of ramp design on ramp vehicle operating characteristics, insight may also be gained through the study of freeway vehicle performance. For example, a clear change in speed characteristics is observed on the freeway lanes adjacent to poorer ramp designs but not adjacent to "good" ramp designs. As indicated in Figure 4.9, the Richmond Avenue right-freeway-lane speed characteristics are smooth and unchanging upstream and downstream of the ramp gore. Under all studied volume combinations, the freeway right lane demonstrates no notable change in the mean speed.

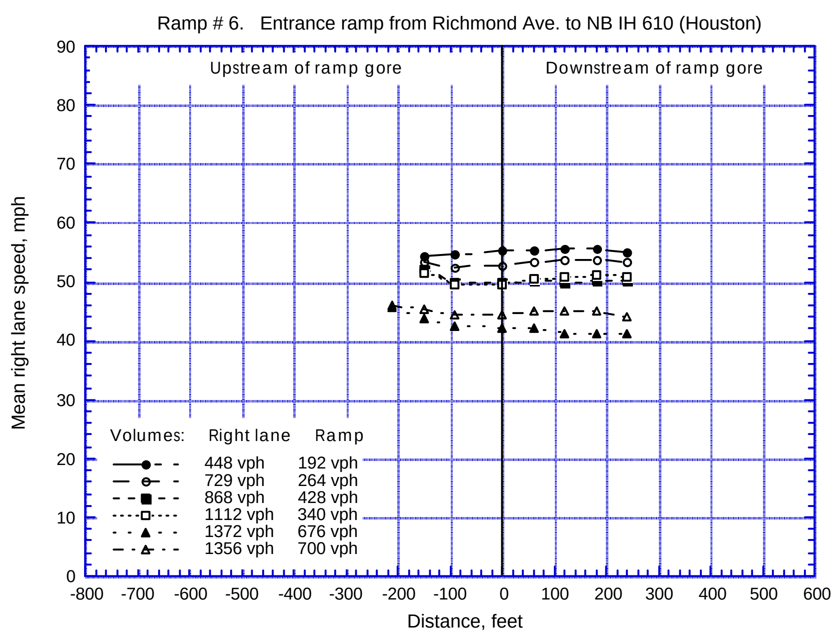


Figure 4.9 Mean Freeway Right Lane Speed, Richmond Avenue to NB IH 610, Houston, Texas

In addition, under most volume combinations the adjacent ramp seems to only minimally impact the mean speed standard deviation. The well-designed San Felipe Road ramp mirrors these general trends. Again, there is little observable influence on the freeway right-lane speeds, with speed characteristics seemingly relatively constant in both the mean and standard deviation. Completely different speed observations are seen on the freeway lane adjacent to the Oltorf Street ramp. As vehicles travel downstream there is higher variability in speed along the roadway and among the different volume combinations.

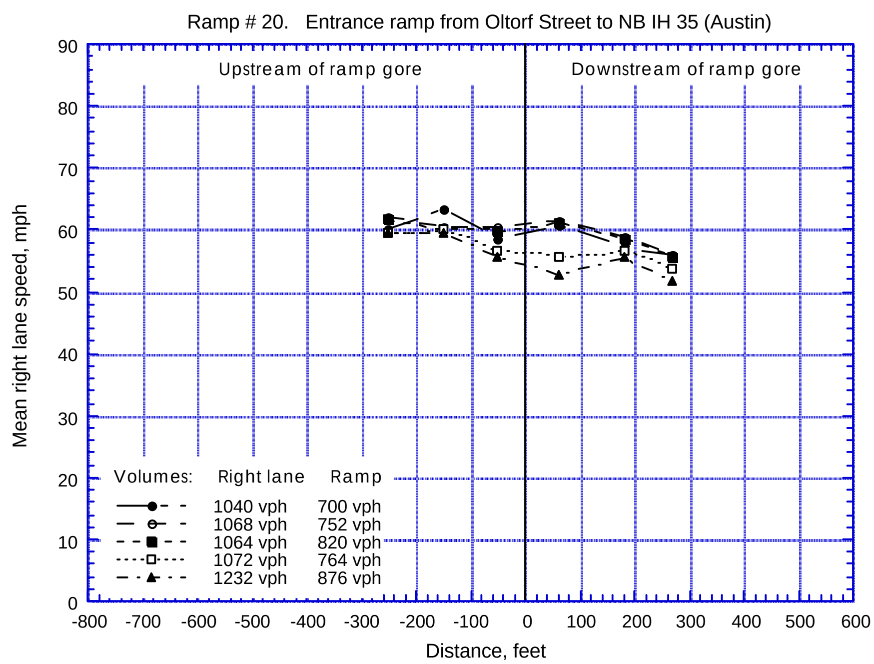


Figure 4.10 Mean Freeway Right Lane Speed, Oltorf Street, Austin, Texas

The general trend is that of decreasing mean speed. This trend is even more pronounced when considering the 15th percentile speeds. In addition, the mean speed standard deviation for all volume combination increases as one travels downstream. The Airport Boulevard ramp further demonstrates these poor ramp design impacts, with decreasing speed and increasing standard deviation exhibited as one travels downstream.

Again, these trends are even more severe in the 15th speed percentile. Unfortunately, the remaining two ramps under study in this report do not provide any meaningful insight into freeway right-lane operations. The 38th Street ramp exhibited low ramp volumes (from 296 to 384 vph) during the periods under study. Such low volume leads to a masking of any impacts on freeway vehicle characteristics, as only overall statistics are considered in this section and relatively few freeway vehicles interact with the ramp vehicles. While the Haskell Road ramp does exhibit more substantial volumes, only three speed measurements could be taken along the right-lane, leading to a difficulty in drawing any meaningful trend or operating characteristics conclusions.

As with the ramp vehicle speeds, the effects due to design on highway vehicle speed may be considered in two parts, upstream and downstream of the ramp gore. The primary effects seen in the ramps studied are on the portion of the freeway lanes downstream of the gore. The poorer ramp designs were accompanied by increasing speed standard deviations and decreasing speeds. For the ramps studied, the speed trends upstream of the gore are not substantially different between the "good" and "bad" ramps. Upstream speed effects could become notable under higher freeway volume conditions, where the speed disturbances would propagate upstream. As discussed in the initial chapters of this report, these high volume periods were eliminated from the current analysis procedure.

Freeway Right-Lane Acceleration and Deceleration *(Figures 9–13, Appendices A–F)*

Along with the freeway right-lane speed characteristics, the acceleration/deceleration characteristics demonstrate clear differences between "good" and "bad" ramps. Both the Richmond Avenue and San Felipe Road ramps display consistent mean acceleration/deceleration rates along the ramps and among the different volume combinations. This pattern is exemplified in Figure 4.11.

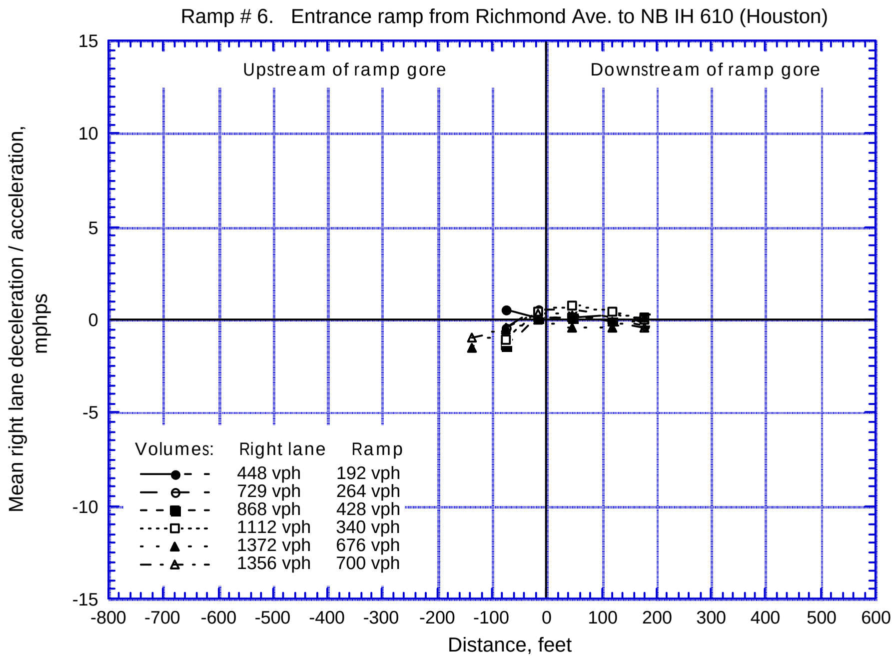


Figure 4.11 Mean Freeway Right Lane Acceleration/Deceleration, Richmond Avenue to NB IH 610, Houston, Texas

The mean acceleration/deceleration rates range between approximately -2 to 1 mphps, with the 85th and 15th percentile falling within a range of roughly -3.5 to 3.5 mphps. The maximum difference between any two consecutive mean acceleration/deceleration points does not exceed approximately 1.5 mphps. The Oltorf Street ramp displays high variability both along the ramp and between the different volume combinations. There are no observable trends among the different volume combinations that suggest any level of predictability in acceleration/deceleration. As shown in Figure 4.12, this variability is observed both upstream and downstream of the gore under all volume conditions.

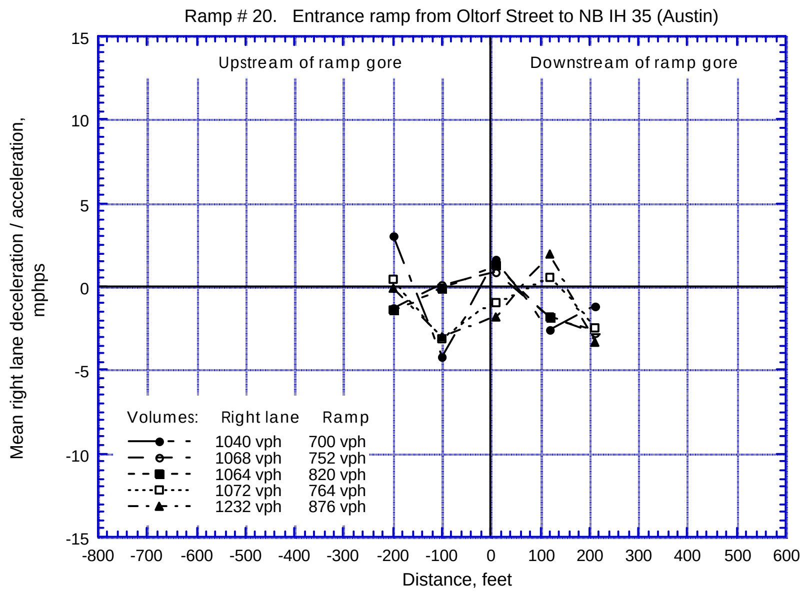


Figure 4.12 Mean Freeway Right Lane Acceleration/Deceleration, Oltorf Street to NB IH 35, Austin, Texas

The mean acceleration/deceleration rates range from roughly -4 to 3.5 mphps, with differences of up to 7 mphps between consecutive data points. The 15th and 85th percentile acceleration/deceleration rates also display the same type of highly variable, inconsistent operations, with rates ranging within -7 to +6 mphps. Differences of up to 8 mphps between consecutive data points are observed. While not as severe as the Oltorf operations, the 38th Street and Airport Boulevard ramps also display a wavelike form in their acceleration/deceleration values as one traverses the ramp length. Also, there is again a lack of a consistent trend between the different volume scenarios on the same ramp. As mentioned in the speed discussion, because the effect of ramp vehicles is somewhat masked on these ramps owing to the relatively low ramp volumes, one would expect poorer performance only as volumes increase. Also as in the speed discussion, no meaningful conclusion may be drawn from the Haskell Road ramp owing to a lack of freeway data points.

So, again, the effect of ramp design on vehicle operating characteristics is clearly witnessed. The poorer the ramp design, the more variable and unpredictable the freeway operations, while for "good" ramp design, under all studied traffic volume ranges the freeway traffic adjacent to the ramp still behaves in a smooth and predictable manner. The observation of variable operation prior to the gore is additionally disturbing in that this indicates that poor ramp design affects the freeway merge area and has effects that propagate up the traffic stream.

Headway *(Figures17–20, Appendices A–F)*

Of the operating characteristics studied, the right-lane freeway headways experienced the least notable influence owing to ramp design. As shown in Figures 4.13 and 4.14, the primary factor in the distribution of headways seems to be the freeway volume.

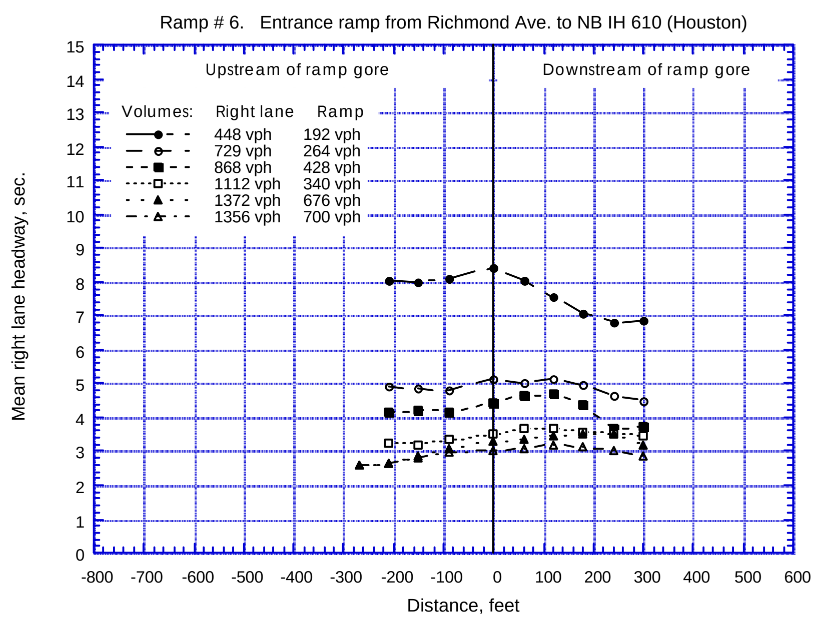


Figure 4.13 Mean Time Headway Freeway Right Lane, Richmond Avenue to NB IH 610, Houston, Texas

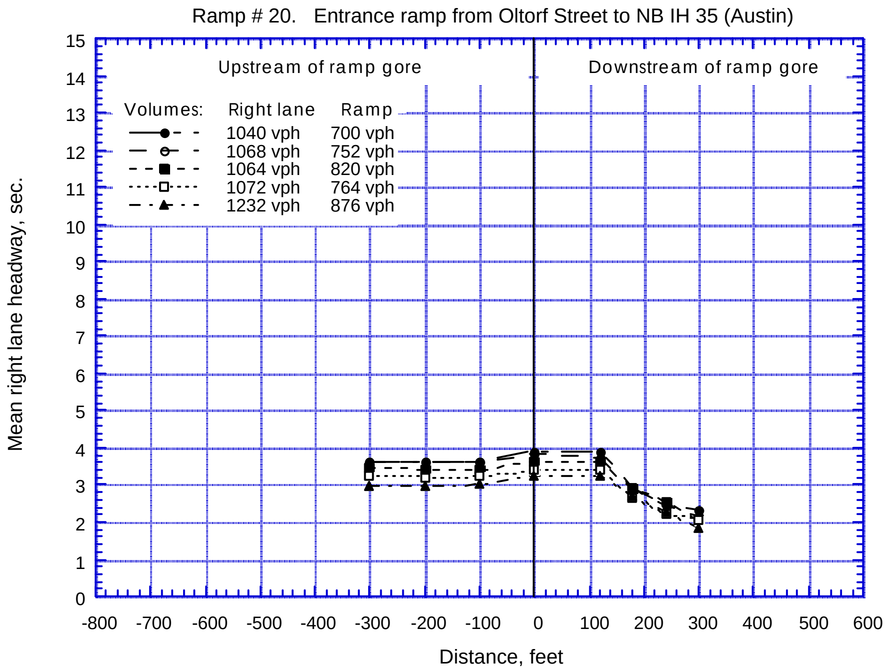


Figure 4.14 Mean Time Headway Freeway Right Lane, Oltorf Street to NB IH 35, Austin, Texas

Different ramp designs under similar volume conditions exhibited similar freeway right-lane headway characteristics. Headways in and of themselves do not lead to potential distinctions between the vehicle operations under different ramp designs, although the importance of this trend in similar headways among like ramp volumes will be seen in the discussion of accepted gaps.

One point worth noting is that the 15th percentile headway for all the ramps consistently maintained an approximately 1 second level. While the mean and 85th percentiles varied between the different volume conditions, the 15th percentile headway did not, with the only exception being larger headways under the lowest volume conditions. Thus, this study has also revealed the minimum headways that will be accepted by drivers.

Accepted Time Gap *(Figures27–30 Appendices A–F)*

An initial, seemingly reasonable hypothesis was that a poorly designed ramp would likely force vehicles to accept smaller gaps. Interestingly, when reviewing the accepted gap trends among the six comparison ramps, it is observed that this does not seem to occur. There is no appreciable difference between the mean, 15th, or 85th percentile accepted gaps on the different ramps. The factor that appears to primarily influence accepted gap is right-lane volume, not ramp design (Figure 4.15).

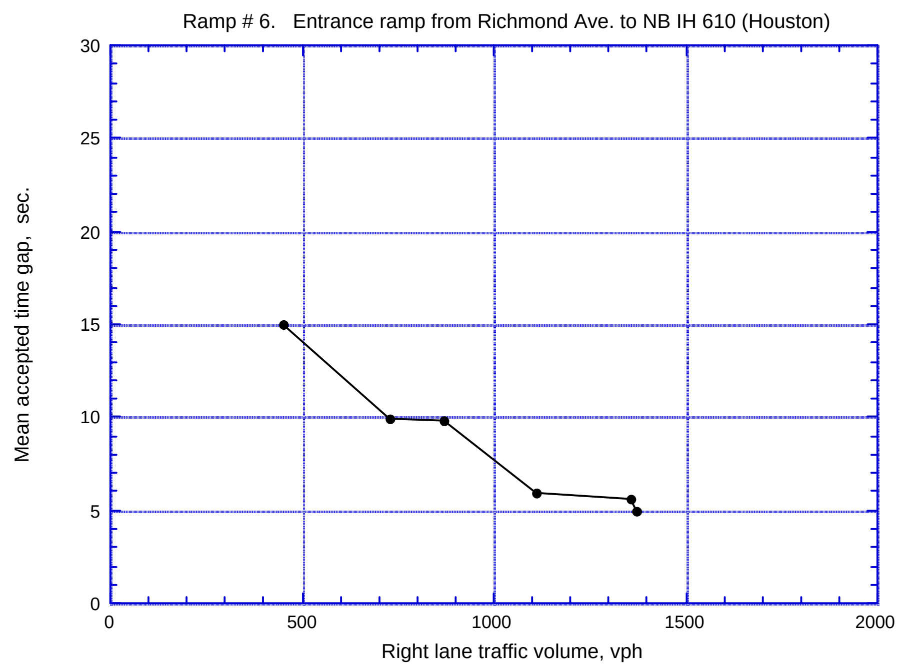


Figure 4.15 Mean Freeway Time Gap Accepted by Ramp Vehicles Versus Right Freeway Lane Traffic Volume, Richmond Avenue to NB IH 610, Houston, Texas

Upon consideration and with respect to the previous discussions in this chapter, this finding becomes a reasonable and expected outcome. First, recall that it has been shown that ramp design has little observable effect on headway distribution, with volumes being the dominant factor. By definition, the accepted gaps are simply a headway sampling; that is, the accepted gap of a vehicle is the headway between that vehicle's lag and lead vehicle conditions. Thus, regardless of the ramp design, the available selection of gaps is similar for similar volumes.

This observation is not intended to imply that accepted gap is an unimportant factor in the consideration of ramp design and driver behavior; it implies only that to understand the effect of accepted gap it is necessary to consider its impact on other ramp vehicle operational characteristics. Drivers are observed seeking and accepting similarly sized gaps. The difference between "good" and "bad" ramps is that the process required to successfully merge into the desired gap on the poorly designed ramp is more difficult. A well-designed ramp allows a driver to smoothly enter a gap and to make necessary speed changes sooner and over longer distances, with smaller acceleration or decelerations. For a driver maneuvering into a similarly sized gap on a poorly designed ramp, reduced time and space in which to perform the maneuvers results in more aggressive actions being necessary to position the vehicle for a successful merge. These types of behavior differences are clearly highlighted in the earlier speed and acceleration discussions.

A method by which this hypothesis may be further confirmed would be to analyze merging vehicle accidents. If this hypothesis were correct, one would expect that many ramp area initial accidents would involve two ramp vehicles. A leading ramp vehicle finding no acceptable gap risks collision with a following ramp vehicle when the lead vehicle initiates some type of slowing or stopping maneuver. Casual observations of such maneuvers are readily made by the common occurrence of vehicles stopping on entrance ramps where the design would typically be considered "bad." Building on this hypothesis, few ramp/highway vehicle initial accidents would be expected. The likely cause of such an accident would be the ramp vehicle attempting to make a forced merge into a gap that is too small. The stated hypothesis leads to the expectation that most drivers would not attempt such a maneuver. As a final note, any such accident analysis must be careful to distinguish between initial and secondary accidents. Once an incident has occurred it is highly likely that other ramp and freeway vehicles may become involved; these incidents are secondary accidents not directly related to the cause of the initial accident.

Merge Location

As indicated in Figure 4.16, larger percentages of merge maneuvers are completed further along the speed change lane as volumes increase. Under the largest traffic volumes, most merges are completed near the end of the 300 ft of monitored speed change lane length on the Richmond ramp.

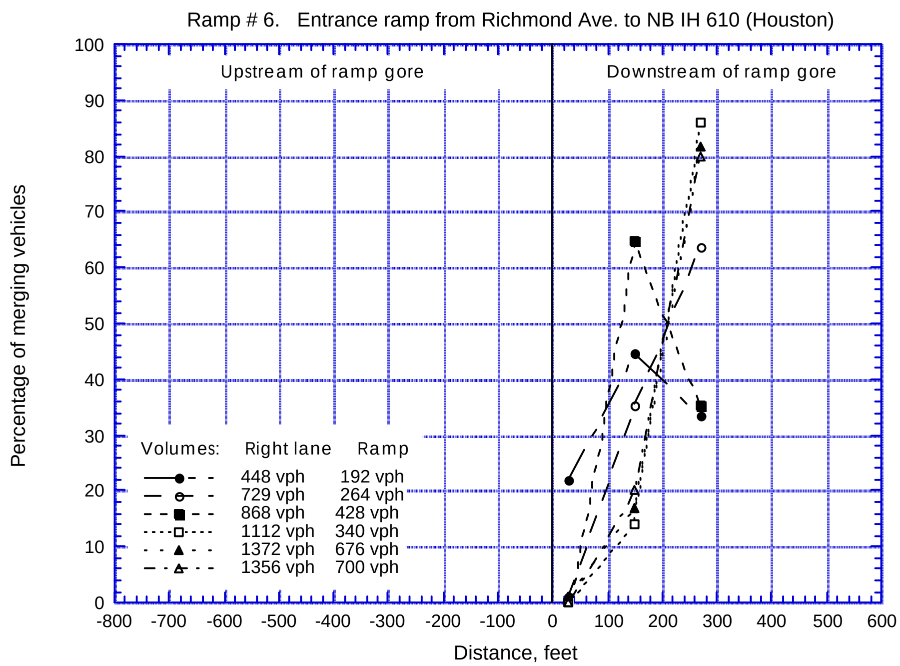


Figure 4.16 Ramp Vehicle Merging Location Percentage,
Richmond Avenue to NB IH 610, Houston, Texas

However, as indicated in Figure 4.17, under similarly high volume conditions, a large fraction of Oltorf ramp users attempt to merge long before the ramp ends. This likely contributes to the effects on freeway main lane speeds and reflects erratic, uncertain driver behavior associated with the lack of sight distance and a short speed-change lane.

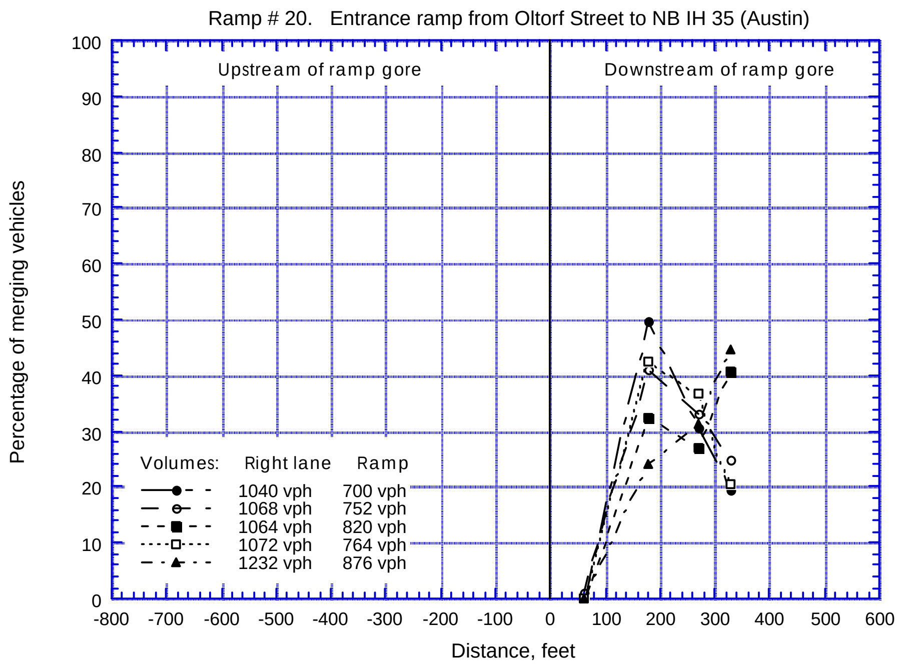


Figure 4.17 Ramp Vehicle Merging Location Percentage,
Oltorf Street to NB IH 35, Austin, Texas

CONCLUSIONS

The analyses of this chapter lead to the following conclusions:

1. Ramp driver speeds on all observed entry ramps are consistently greater than 50 percent of the freeway design speed. Therefore, a design criterion specifying a ramp design speed to be 50 percent of the freeway design speed is questionable.
2. Ramp driver speed-distance histories on ramps with “good” geometrics (i.e., adequate sight distance and speed-change lane lengths) exhibit smooth

appearances and no abrupt speed changes. Speed-distance histories for ramp-freeway facilities with vertical profiles limiting ramp driver sight distance and marginal speed change lane lengths exhibit undulating waveforms, indicating significant ramp driver speed changes.

3. Acceleration/deceleration rate histories for ramps with “good” and “bad” geometrics have patterns similar to speed histories. “Good” geometrics typically produce small positive acceleration rates (0 to 2 mphps), while “bad” geometrics produce larger values of positive and negative acceleration (-4 to +4 mphps).
4. Freeway right lane speeds are not largely affected by ramp vehicles if the ramp has “good” geometrics (as characterized here). “Bad” geometrics tend to cause significant reductions of freeway right-lane speeds, particularly under high freeway and ramp traffic volumes.
5. Freeway right-lane time headways tend to be influenced not by ramp design complexity, but rather by traffic volume.
6. The factor that seems to primarily influence the size of time gap accepted by merging ramp drivers is freeway right-lane volume, not ramp design.
7. Under high traffic volume conditions, most ramp drivers on a ramp having “good” geometrics travel to the end before merging. If geometrics are poor, this trend is much less pronounced, as drivers aggressively merge from any location to avoid being trapped at the speed-change lane end.

CHAPTER 5. ANALYTICAL MODELING

Analyses of experimental data described in the previous chapter provide elements of a freeway entry ramp driver behavioral characterization. Efforts of this chapter are designed to develop conceptual models. It presents several approaches to modeling ramp driver acceleration/deceleration behavior. The importance of the ramp driver's ability to see freeway traffic before reaching the ramp gore is discussed and, finally, a change to the AASHTO acceleration length model is suggested.

ACCELERATION-DECELERATION

One approach to acceleration/deceleration modeling is prediction of rates for individual vehicles during successive small time or distance increments. This approach led to the development and calibration of a very sophisticated nonlinear model as part of a Southwest Region University Transportation Center (SWUTC) study [25]. This framework incorporates a series of dummy variables to generalize the model specification. The framework, as specified in Equation 5.1, was designed to model ramp vehicle acceleration-deceleration behavior under all possible freeway merge situations.

$$
\begin{aligned}
\ddot{X}_{r_i}\left(d_j + D\right) = {} & \beta_0 \\
& + \delta_1 D_{1j} + \beta_1 D_{1j} \dot{X}_{r_i}\left(d_j + D\right)^{\gamma} \frac{\dot{X}_{flag_i}\left(d_j\right) - \dot{X}_{r_i}\left(d_j\right)}{\left[\left(X_{r_i}\left(d_j\right) - X_{flag_i}\left(d_j\right)\right)^2 + W^2{}_{rflag_i}\left(d_j\right)\right]^{\alpha_1}} \\
& + \delta_1 D_{2j} + \beta_2 D_{2j} \dot{X}_{r_i}\left(d_j + D\right)^{\gamma} \frac{\dot{X}_{r_i}\left(d_j\right) - \dot{X}_{flead_i}\left(d_j\right)}{\left[\left(X_{flead_i}\left(d_j\right) - X_{r_i}\left(d_j\right)\right)^2 + W^2{}_{rflead_i}\left(d_j\right)\right]^{\alpha_2}} \\
& + \delta_3 D_{3j} + \beta_3 D_{3j} \dot{X}_{r_i}\left(d_j + D\right)^{\gamma} \frac{\dot{X}_{r_i}\left(d_j\right) - \dot{X}_{rlead_i}\left(d_j\right)}{\left[\left(X_{rlead_i}\left(d_j\right) - X_{r_i}\left(d_j\right)\right)^2 + W^2{}_{rrlead_i}\left(d_j\right)\right]^{\alpha_3}} \\
& + \delta_4\left(1 - D_{3j}\right) D_{4j} + \beta_4\left(1 - D_{3j}\right) D_{4j} \dot{X}_{r_i}\left(d_j + D\right)^{\gamma} \frac{\dot{X}_{r_i}\left(d_j\right)}{\left[L - \left(X_{r_i}\left(d_j\right)\right)^2 + W^2{}_{rend_i}\left(d_j\right)\right]^{\alpha_4}} \\
& + u_{r_i d_j}
\end{aligned}
$$

(5.1, Ref 25)

where:

$\ddot{X}_{r_i}\left(d_j + D\right)$: acceleration rate of ramp vehicle i at location d_j+D;

$X_{r_i}(d_j)$: location of ramp vehicle i when it passed fiducial mark j; alternatively, it is the location of fiducial mark j measured from the merging end, j=1, 2,..., m_i;

$X_{flag_i}(d_j)$: location of ramp vehicle i's corresponding freeway lag vehicle when vehicle i passed fiducial mark j;

$X_{flead_i}(d_j)$: location of ramp vehicle i's corresponding freeway lead vehicle when vehicle i passed fiducial mark j;

$\ddot{X}_{r_i}\left(d_j\right)$: velocity of ramp vehicle i when it passed fiducial mark j; alternatively, it is the velocity of ramp vehicle i at location d_j;

$\ddot{X}_{flag_i}\left(d_j\right)$: velocity of ramp vehicle i's corresponding freeway lag vehicle when vehicle i passed fiducial mark j;

$\ddot{X}_{flead_i}(d_j)$: velocity of ramp vehicle i's corresponding freeway lead vehicle when vehicle i passed fiducial mark j;

$u_{r_i d_j}$: disturbance of the estimated acceleration rates for the observation of ramp vehicle i when it passed fiducial mark j;

d_j : position of fiducial mark j measured from the merging end;

L : length of the acceleration lane;

D : distance lag;

m_i : the total number of observations of estimated acceleration-deceleration rates of ramp vehicle i.

$X_{rlead_i}(d_j)$: location of ramp vehicle i's corresponding ramp lead vehicle when vehicle i passed fiducial mark j;

$\dot{X}_{rlead_i}(d_j)$: velocity of ramp vehicle i's corresponding ramp lead vehicle when vehicle i passed fiducial mark j;

$W_{rflag_i}(d_j)$: lateral offset between ramp vehicle i and its corresponding freeway lag vehicle when vehicle i passed fiducial mark j;

$W_{rflead_i}(d_j)$: lateral offset between ramp vehicle i and its corresponding freeway lead vehicle when vehicle i passed fiducial mark j;

$W_{rrlead_i}(d_j)$: lateral offset between ramp vehicle i and its corresponding ramp lead vehicle when vehicle i passed fiducial mark j;

$W_{rend_i}(d_j)$: lateral offset of ramp vehicle i relative to the ramp end when vehicle i passed fiducial mark j;

D_{1j}, D_{2j}, D_{3j}, and D_{4j} are dummy variables defined as follows:

1 if there are freeway lag vehicles when vehicle i passed fiducial mark j

D_{1j} = 0 otherwise

1 if there are freeway lead vehicles when vehicle i passed fiducial mark j

D_{2j} = 0 otherwise

1 if there are ramp lead vehicles when vehicle i passed fiducial mark j

D_{3j} = 0 otherwise

1 if ramp vehicle is within 300 feet to the ramp end when vehicle i passed fiducial mark j

D_{4j} = 0 otherwise

and β_0, β_1, β_2, β_3, β_4, α_1, α_2, α_3, α_4, δ_1, δ_2, δ_3, δ_4, and γ are parameters to be estimated.

Although this approach proved to work reasonably well for ramps having geometrics meeting current design criteria, it could not generally replicate the erratic waveform acceleration patterns observed on ramps having substandard sight distance and speed change lanes.

CONTINUOUS ACCELERATION-DECELERATION MODEL

Analyses conducted during the same SWUTC study indicated a strong relationship between acceleration/deceleration rate and vehicle speed. Recalling that acceleration, by definition, is the time-rate-change of speed, one should not be surprised by that result. Consequently, one might expect fairly reliable acceleration-deceleration rate estimations using instantaneous vehicle speed as an explanatory variable. Scatter plots of acceleration and deceleration rates versus ramp vehicle speeds are shown in Figures 5.1 and 5.2, respectively [25]. In Figure 5.1, a family of two distinct acceleration curves is apparent, while in Figure 5.2, three distinguishable curves are clearly seen. The families of curves indicate that drivers having the same speeds adopt different acceleration or deceleration rates. This phenomenon might be caused by vehicle performance, by driver aggressiveness, or by other unobservable driver vehicle factors. In other words, as stimuli change, a ramp driver, even though running at the same speed, might use different acceleration or deceleration rates. For practical applications, this randomness can be accommodated by introducing probabilistic random numbers to determine which curve should be applied to estimate acceleration-deceleration rate magnitudes.

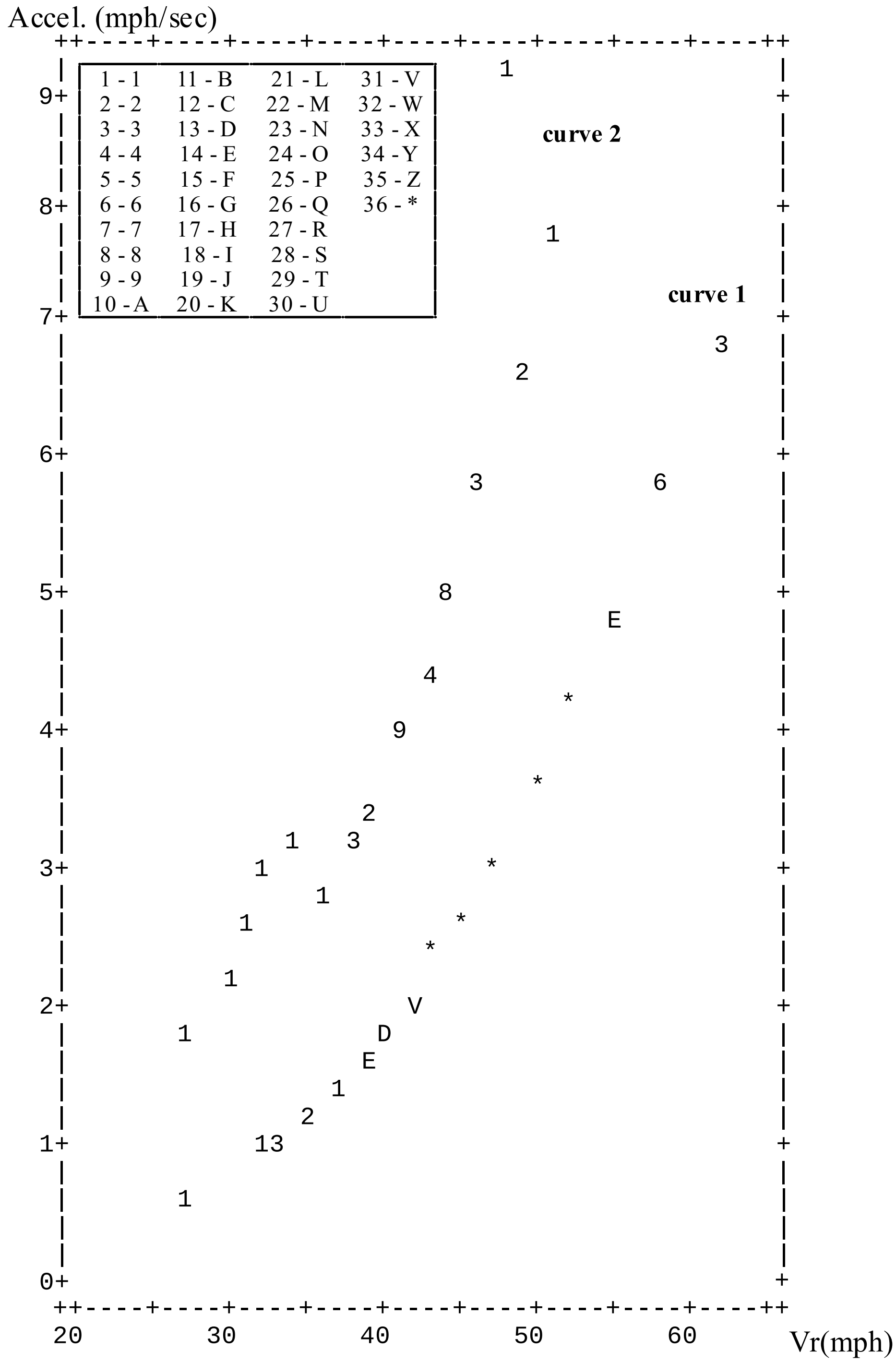


Figure 5.1 Acceleration Versus Vehicle Speed [25]

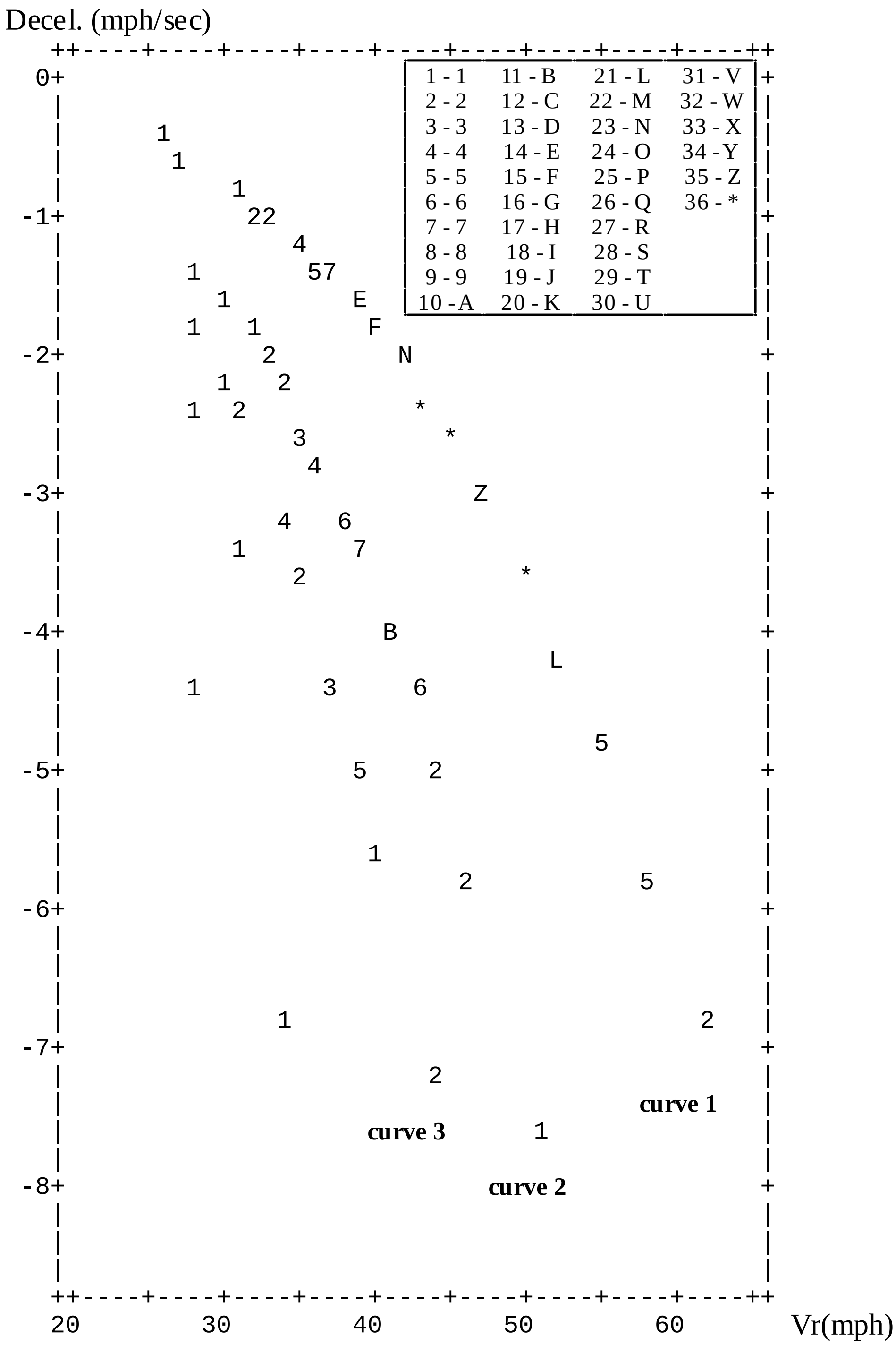


Figure 5.2 Deceleration Versus Vehicle Speed [25]

Based on the scatter plots of Figures 5.1 and 5.2, hypothetical expressions of continuous acceleration and deceleration models are shown as follows:

Acceleration model

$$\text{Accel} = e^{a + bV_r} \tag{5.2}$$

Deceleration model

$$\text{Decel} = -e^{a + bV_r} \tag{5.3}$$

The parameters *a* and *b* were estimated for each of the five curves using simple transformations and conventional linear regression procedures. Results shown in Table 5.1 are promising, with significant coefficients in all parameters and high adjusted R-squared values.

Table 5.1 Acceleration/Deceleration Model Parameters

Model	Parameters		
Acceleration	a	b	R-squared
Curve 1	-1.99	0.065	0.995
Curve 2	-1.04	0.06	0.958
Deceleration			
Curve 1	-2.13	0.068	0.994
Curve 2	-1.83	0.078	0.996
Curve 3	-0.99	0.067	0.841

The need for three deceleration curves is indicative of more variability in deceleration rates, which might be owing to the fact that ramp drivers do not really intend to decelerate during freeway merge maneuvers. The calibrated models should not be extrapolated beyond ramp vehicle speeds lower than 25 mph or greater than 65 mph, since that is the observed data range.

A logit binary choice model was developed for predicting whether a driver would choose to accelerate or decelerate during any analysis increment [25]. However, as with the previously presented approach (Equation 5.1), such a model is not totally successful when applied to ramps having substandard geometrics. Therefore, the development is noted here but no detailed presentation is offered.

ACCELERATION RATE DESIGN VALUE MODEL

The previous discussions examined theoretical acceleration/deceleration models. A primary study issue, however, is whether current AASHTO ramp design acceleration rates are appropriate. Table 5.2 has been developed to address this question by presenting the AASHTO assumed initial and final speeds and recommended acceleration lane lengths, as well as the constant acceleration rates implicitly used. The top half of Table 5.2 is essentially Table X-4 from the current AASHTO policy [1], while the bottom half presents the constant acceleration rates required to permit the stated speed changes within the recommended distances.

Table 5.2 Implied AASHTO Acceleration Rates

Acceleration Length (feet) for Entrance Curve Design Speed (From AASHTO, Table X-4)									
Speed Reached (mph)	Initial Speed (mph)								
	0	14	18	22	26	30	36	40	44
23	190								
31	380	320	250	220	140				
39	760	700	630	580	500	380	160		
47	1170	1120	1070	1000	910	800	590	400	170
53	1590	1540	1500	1410	1330	1230	1010	830	580
Acceleration Rate (mphps) Required for Speed Changes and Distances Shown Above									
Speed Reached (mph)	Initial Speed (mph)								
	0	14	18	22	26	30	36	40	44
23	2.05								
31	1.86	1.76	1.87	1.59	1.50				
39	1.47	1.39	1.40	1.31	1.24	1.20	1.03		
47	1.39	1.32	1.29	1.27	1.24	1.20	1.14	1.12	1.18
53	1.30	1.25	1.22	1.21	1.18	1.14	1.10	1.07	1.11

The table indicates that for initial speeds of 26 mph or more the implied acceleration rates are 1 to 1.5 miles per hour per second and average approximately 1.2 mphps. Since most Texas freeway entry operations are initiated from frontage roads or arterial streets with speeds greater than 26 mph, these are the cases of most interest. Comparison of these implied acceleration rates to those measured during this study could indicate whether drivers actually behave as assumed by the AASHTO policy.

As noted earlier, acceleration rates were measured as speed changes across a series of speed traps typically beginning prior to the entry ramp gore and extending almost to the end of the speed change lane. Resulting acceleration-distance histories of ramp facilities whose geometrics meet AASHTO design guidelines were quite different from those whose geometry does not meet AASHTO criteria. The ramps with poor geometry have oscillating acceleration-distance patterns containing large magnitude positive and negative acceleration values. On the other hand, those with good geometrics tend to have smaller magnitude acceleration values that are almost all positive. Mean, 85th percentile, 15th percentile, and other descriptive statistics were computed for accelerations at each speed trap for each volume condition and each entry ramp. Since design values of many parameters are often associated with an 85th percentile, which is sometimes considered a point of diminishing returns, this statistic was chosen to represent measured accelerations. Because data for each speed trap pair, for each volume condition, and for each ramp yielded a different 85th percentile value, a grand mean of the 85th percentiles for each ramp was computed. To provide a conceptual view of the range of 85th percentile values for each ramp across all volume conditions, maximums and minimums were also identified. As noted previously, acceleration rates for ramps meeting and not meeting AASHTO design criteria differed widely. Therefore, values are presented separately for these two ramp categories in Table 5.3. The table presents data for a sample that includes two of the geometrically best and three of the geometrically worst ramps observed.

The two ramps with “good” geometrics should provide fair assessments of acceleration rates chosen by drivers under typical AASHTO design conditions. Therefore, comparison of the mean 85th percentile acceleration rate with the AASHTO implied rates should be relevant. The mean observed value is approximately 1.6 mphps and the AASHTO mean for initial speeds 26 mph or greater is approximately 1.2 mphps. These values seem to compare favorably, particularly when the variability in observed values is considered as indicated by the maximum of 3.2 and minimum of 0.5 mphps. The differences between the observed and AASHTO values are so small that they do not seem to justify any recommendation for AASHTO value changes. The much larger acceleration rates associated with ramps with poor geometrics are probably indications of drivers feeling forced to use unusual trajectories in order to negotiate problematic freeway entry facilities. Although the larger rates have now been documented, they should not be considered representative of the conditions for which entry ramps should be designed.

Table 5.3 Observed Ramp Driver Acceleration Rates

Geometric Category	Ramp	Grand Mean 85th Percentile Acceleration Rate (mphps)	Maximum 85th Percentile Acceleration Rate (mphps)	Minimum 85th Percentile Acceleration Rate (mphps)
Poor				
	Oltorf to IH 35	2.1	5.2	-1.9
	38th to IH 35	2.7	4.9	0.5
	Haskell to IH 75	2.7	6.1	0.4
	Mean	2.5	5.4	-0.3
Good				
	San Felipe to IH 610	1.8	4.1	0.1
	Richmond to IH 610	1.4	2.3	0.8
	Mean	1.6	3.2	0.5

ENTRY RAMP SIGHT DISTANCE

The AASHTO policy [1] acknowledges the importance of sight distance in ramp design. The policy states:

> *In general, adequate sight distance is more important than a specific gradient control and should be favored in design. Usually, these two controls are compatible . . . . Ramp grades should be as flat as feasible to minimize the driving effort required in maneuvering from one road to another.*

With these statements, the policy seems to be addressing a driver's need to see far enough down a path to be able to stop before or maneuver around potential problems. However, a ramp driver faces an unusually complex task, since the downstream path of an entry ramp driver includes the freeway right lane traffic stream to which merging is intended. Smooth ramp operations seem to be dependent on drivers being able to see the right freeway lane traffic as early in the merging process as possible, but definitely prior to reaching the ramp gore. Grade differences between the ramp driver path and the freeway main lanes, structures, and crash or median barriers can easily block ramp drivers' view of freeway gaps until after the ramp gore.

Figure 5.3 presents the acceleration-distance history of mean acceleration rates for three different entry ramp cases. The Richmond Avenue to northbound IH 610 entry ramp in Houston provides ramp drivers not only with an unobstructed view of freeway traffic several hundred feet prior to the ramp gore, but also with an adequately long speed-change lane. As indicated in the figure, mean acceleration values are continuously positive and of small magnitude. Such values are typical for ramps that have adequate speed-change lane length and that provide drivers an opportunity to examine freeway traffic and to begin gap searching prior to the ramp gore.

The Haskell Road to IH 75 entry ramp in Dallas was a temporary facility in a construction zone; it is nonetheless important for this discussion because, while it provides only slight obstruction to a ramp driver's view of freeway traffic, it lacks adequate speed-change lane length. Construction barricades adjacent to the ramp may have obstructed the view of some drivers, a situation that contributed to the erratic acceleration-distance pattern. Unlike the Richmond Avenue case, the figure indicates that average acceleration values oscillate from positive to negative, as is typical of ramps not meeting design criteria. However, the only negative value, one indicating braking, is of small magnitude and occurs slightly over 100 ft prior to the ramp gore. Thus, drivers respond to the uncertain merging situation by slight braking followed by significant acceleration followed by braking as the merge maneuver is completed.

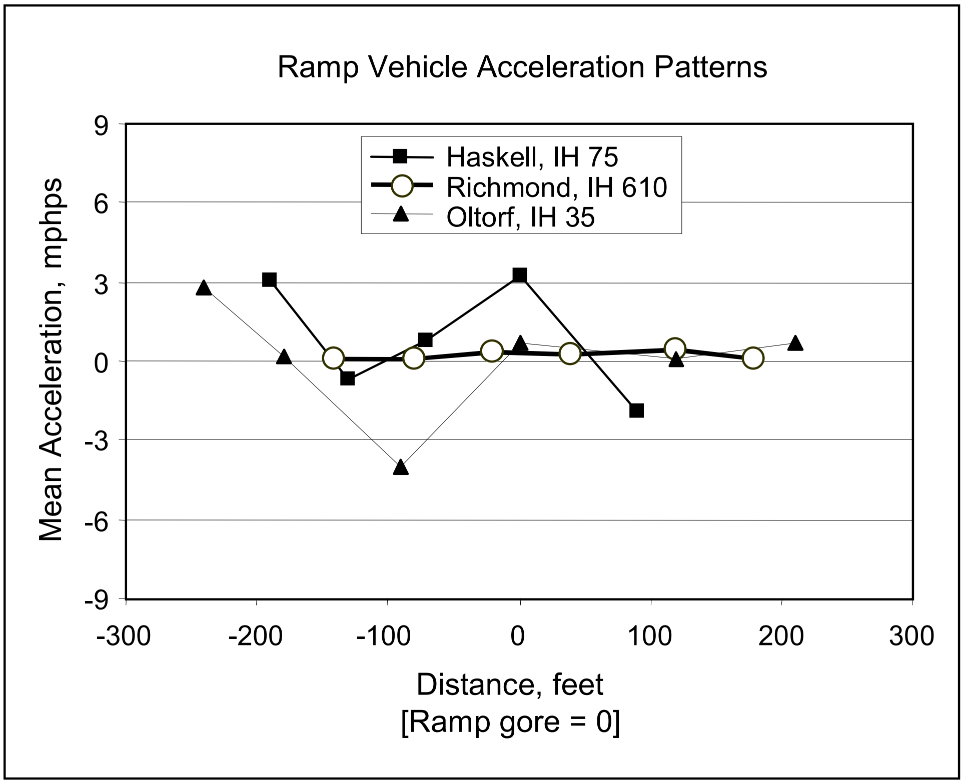


Figure 5.3 Mean Acceleration Versus Distance for Three Special Entry Ramp Cases

Significantly, acceleration begins prior to the ramp gore, a finding that is in agreement with the AASHTO assumption. Figure 5.4 illustrates how the AASHTO policy suggests the speed-change length measurement is to be made for taper-type entry ramps. A version of this drawing appears at the bottom of AASHTO Table X-4, from which the acceleration lengths of Table 5.2 were taken. If the speed changes indicated in Table 5.2 are to be made with the small, comfortable acceleration rates indicated in Table 5.2, the entire acceleration length, L, must be available for the acceleration process. As indicated in Figure 5.3, that is the case for the Richmond Avenue ramp. However, at the Haskell Road ramp, drivers are actually braking about 100 ft prior to the ramp gore, indicating that a continuous, gentle acceleration over the entire acceleration length is not occurring.

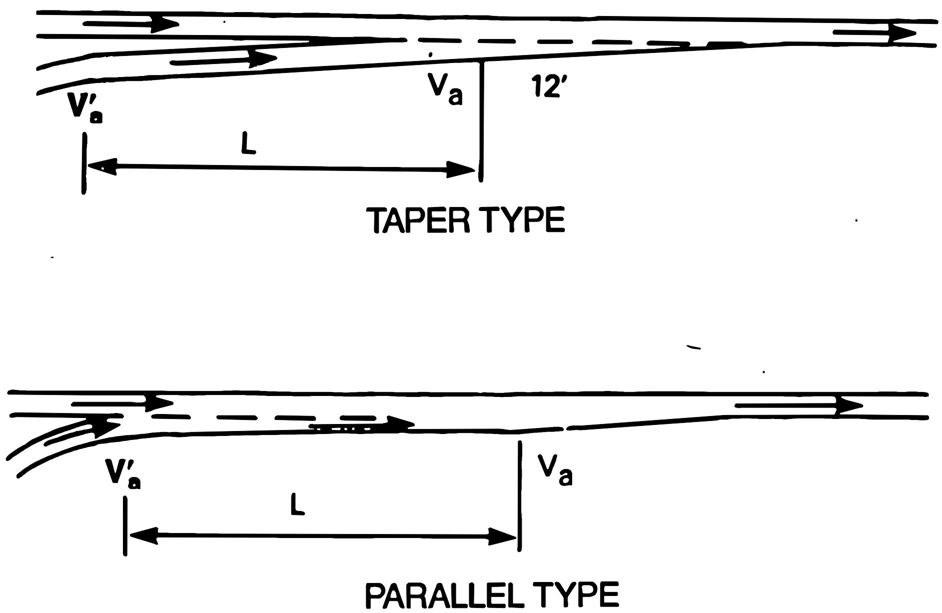


Figure 5.4 Speed Change Lane Length Measurement According to AASHTO Table X-4

The acceleration pattern for the Oltorf Street to northbound IH 35 entry ramp depicts a case in which a driver's view of freeway traffic is completely obstructed until near the ramp gore. This obstruction is due to a difference in grades between the freeway main lanes and the frontage road (a result of area topography). Additionally, the speed change length is less than the AASHTO minimum. Average accelerations shown in Figure 5.3 indicate significant braking prior to the ramp gore as drivers anxiously seek their first clear view of freeway traffic. For many drivers, this braking action results in a stop near the ramp gore. The field crew for this study witnessed a near collision in which one driver stopped suddenly near the gore, forcing another driver, looking toward the freeway, behind the stopped vehicle, to drive around the stopped vehicle onto the grass median to avoid a rear-end collision. Figure 5.3 indicates that positive acceleration begins at or only slightly before the ramp gore.

Therefore, as indicated in Figure 5.4, most of the acceleration length prior to the ramp gore is not really used, resulting in the actually used length being quite short. Thus, the Oltorf entry ramp illustrates how drivers do not use acceleration lane length prior to the ramp gore if their view of the freeway is obstructed.

In Austin, most entry ramps along IH 35 below the upper deck severely limit a driver's view of the freeway prior to the ramp gore. Combinations of upper deck structure and topography typically provide the obstructions. Ramps at Airport Boulevard, 38th 1/2 Street, and 32nd Street, which were videotaped, all produced similar driver performance characteristics. Based on drivers not beginning the acceleration process until after the ramp gore, all provide inadequate speed change lengths.

SUGGESTED MODIFICATION TO AASHTO TAPER-TYPE RAMP ACCELERATION LENGTH MEASUREMENT

The analyses and observations of the previous section seem to lead to a suggested modification of the manner in which acceleration length is measured for taper-type entry ramps. If there are no obstructions to the driver's view of the freeway right lane prior to the entry ramp gore, the current AASHTO acceleration length measurement, as depicted in Figure 5.4, and the AASHTO policy should continue to be used. However, drivers tend not to begin the acceleration process until they have a clear view of freeway right-lane traffic; therefore, designers should begin the length measurement where drivers begin accelerating. If obstructions to the driver's view prior to the ramp gore cannot be removed, the acceleration length should be measured from the first location providing an unobstructed view of the freeway right lane.

SUMMARY

Acceleration models depicting entry ramp speed changes were examined from both theoretical and empirical points of view. Observations of traffic operations on ramps meeting current design criteria indicate that drivers use acceleration rates similar to those used by the current AASHTO policy. Differences between observed and AASHTO rates were not sufficiently great to justify a recommended change.

Acceleration lengths for taper-type entry ramps should include only the lane portions from which ramp drivers can clearly view the freeway right-lane traffic.

CHAPTER 6. CONCLUSIONS AND RECOMMENDATIONS

The observations and analyses described in the preceding five chapters have led to the following series of conclusions and recommendations:

1. Average ramp driver speeds on all observed entry ramps were consistently greater than 50 percent of the freeway design speed, even where the freeway design speed was 70 mph. Thus, designing entry ramps for speeds lower than the speeds at which drivers typically operate is inappropriate**.**

   Recommendation: The design criterion allowing an entrance ramp design speed to be 50 percent of the freeway design speed should be deleted from AASHTO and TxDOT policy.

2. Ramp driver speed-distance history plots for ramps with adequate sight distance and speed change lane lengths exhibited smooth appearances and indicated no abrupt speed changes. Speed-distance plots for vehicles operating on ramp-freeway facilities with vertical profiles that limited ramp driver sight distance and had marginal speed change lane lengths exhibited undulating waveforms, indicating significant ramp driver speed changes.

3. Acceleration/deceleration rate versus distance along the ramp plots for ramps with "good" and "bad" geometrics had patterns similar to the speed distance plots. Ramps having adequate sight distance and speed change lane lengths typically produced small positive acceleration rates (0 to 2 mphps). The observed values were comparable to the implied rates contained in AASHTO Table X-4 (acceleration lengths). Ramps with inadequate sight distance and/or inadequate acceleration lane lengths produced larger values of positive and negative acceleration (-4 to +4 mphps).

Recommendation: The AASHTO acceleration rate model used to estimate acceleration lane lengths should not be changed.

4. Freeway right-hand lane speeds were not largely affected by ramp vehicles where the ramp had adequate sight distance and speed change lane lengths. Inadequate sight distance and/or inadequate acceleration lane lengths tended to cause significant reductions of freeway right-hand lane speeds, particularly under high freeway and ramp traffic volumes.

5. Freeway right-hand lane time headways seemed to be influenced not by complex ramp design features, but rather by traffic volume.

6. Freeway right-hand lane volume appeared to be the primary factor influencing the size of time gap that was accepted by merging ramp drivers.

7. Under high traffic volume conditions, most ramp drivers traveled nearly to the end of the ramp with adequate sight distance and speed change lane length before merging smoothly into the freeway lane. If a ramp had inadequate sight distance and/or inadequate acceleration lane length, drivers more aggressively merged immediately beyond the entrance ramp gore to avoid being trapped at the end of the speed change lane.

8. Drivers tended to begin the concurrent acceleration/merge process only after gaining a clear view of freeway right lane traffic. If grades, structures, or barricades obstructed the ramp driver's view of the right-hand lane freeway, acceleration did not begin until near the ramp gore where the view became unobstructed. The AASHTO acceleration length model for tapered entrance ramp terminals features a large fraction of the acceleration length prior to the ramp

gore. If the driver's view is obstructed prior to the ramp gore, the AASHTO model can incorrectly represent available acceleration length.

Recommendation: Acceleration lengths for taper type entry ramps should include only the lane portions from which ramp drivers can clearly view vehicles in the right-hand freeway lane. The AASHTO acceleration length model should be clarified to include this additional stipulation.

9) As noted in Chapter 1 and in Reference 24, at lower design speeds the TxDOT design standard may provide for a taper length shorter than that in the AASHTO standard. This difference stems from the method of inclusion of the taper length from full lane width to lane elimination.

Finally, as noted in Chapters 4 and 5, driver behavior upstream and downstream of the ramp gore may exhibit different characteristics. A design procedure that allows for flexibility in choosing separate design speeds upstream and downstream of the gore may provide for overall superior designs. For example, the current 50 percent criteria produces a very desirable long acceleration lane but permits speed-limiting horizontal and vertical alignment elements upstream of the ramp gore. Provision of a high design speed for upstream features and a low design speed for downstream features may provide for an optimal design standard. While this item is not listed as a recommendation, as it somewhat exceeds the scope of this research, it would seem that this effort leads to a belief that such a broad change in design philosophy should be considered, studied, and potentially implemented.

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APPENDIX A

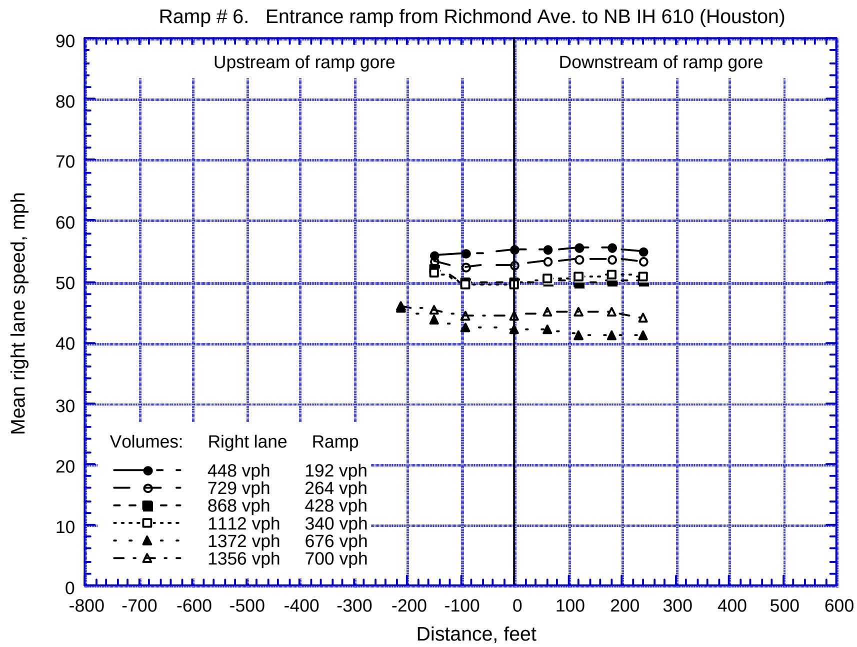

Figure A01 Mean Freeway Right Lane Speed, Ramp #6,
Entrance Ramp from Richmond Avenue to NB IH 610, Houston

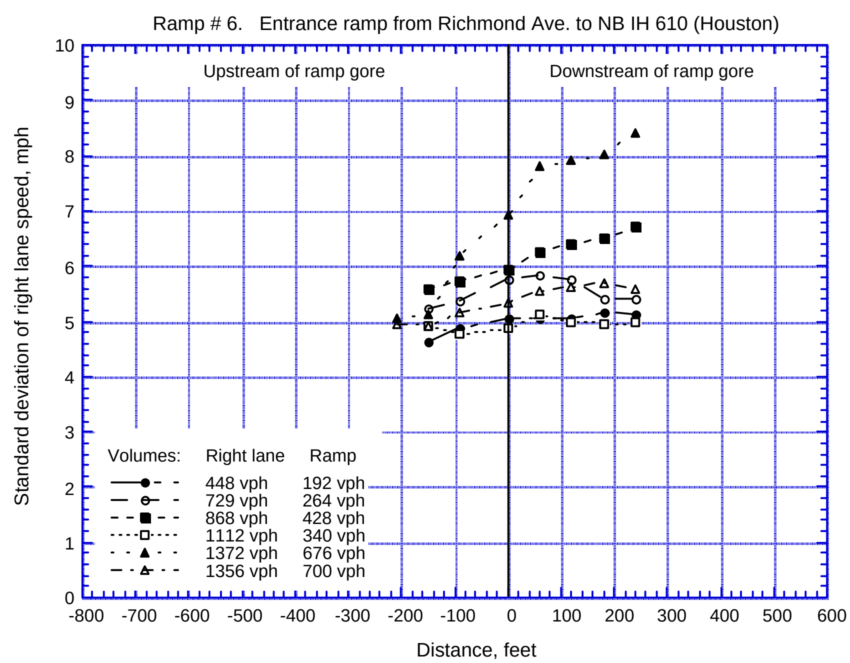

Figure A02 Standard Deviation of Freeway Right Lane Speed, Ramp #6,
Entrance Ramp from Richmond Avenue to NB IH 610, Houston

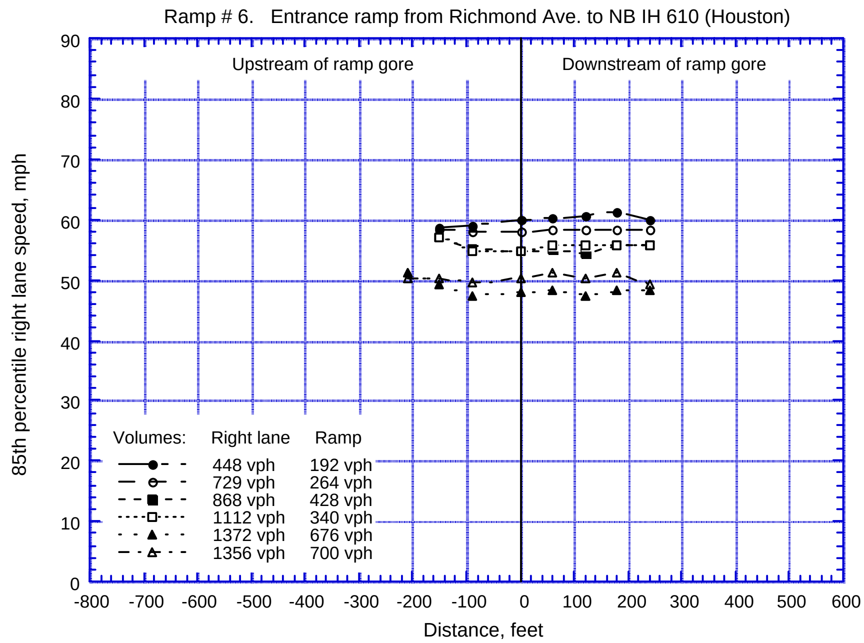

Figure A03 85th Percentile Freeway Right Lane Speed, Ramp #6, Entrance Ramp from Richmond Avenue to NB IH 610, Houston

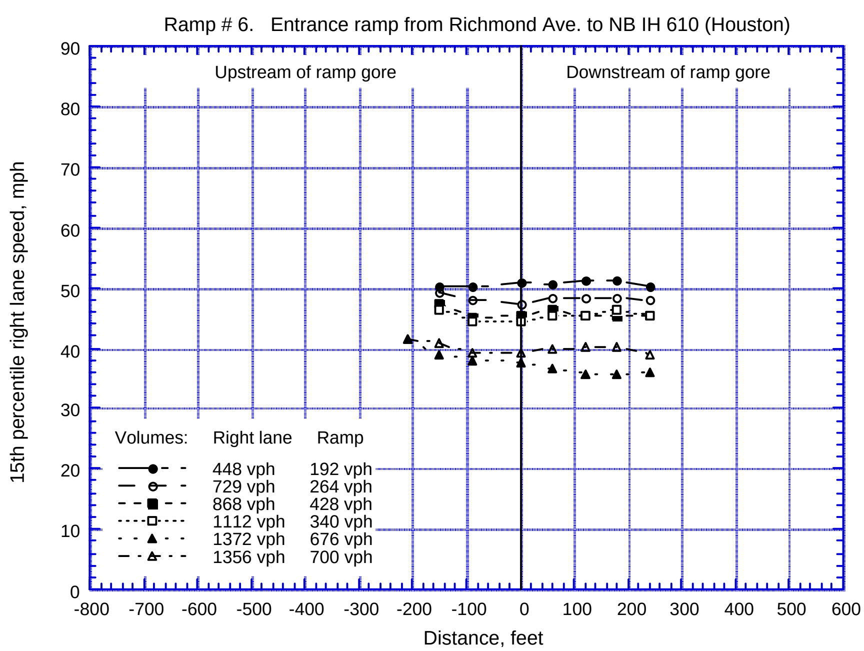

Figure A04 15th Percentile Freeway Right Lane Speed, Ramp #6, Entrance Ramp from Richmond Avenue to NB IH 610, Houston

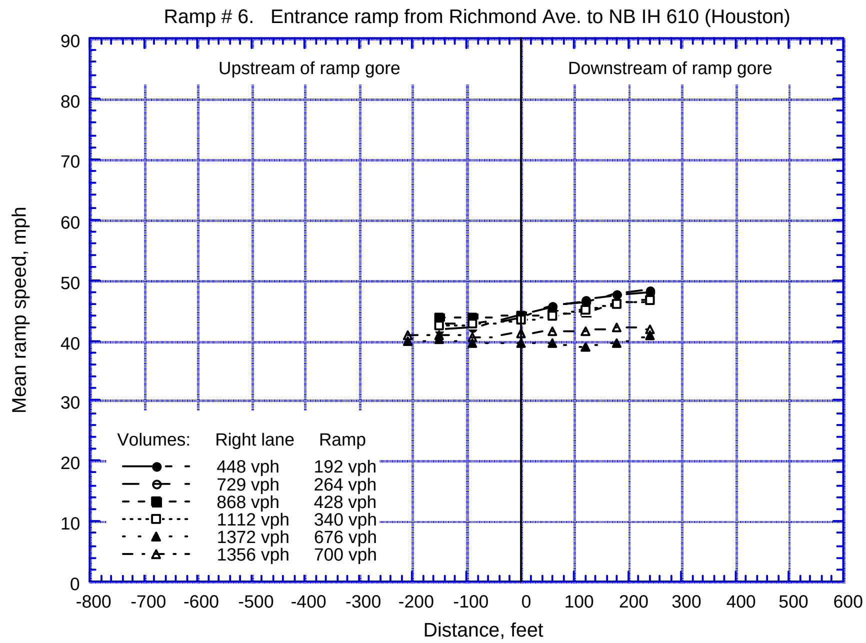

Figure A05 Mean Ramp Speed, Ramp #6, Entrance Ramp from Richmond Avenue to NB IH 610, Houston

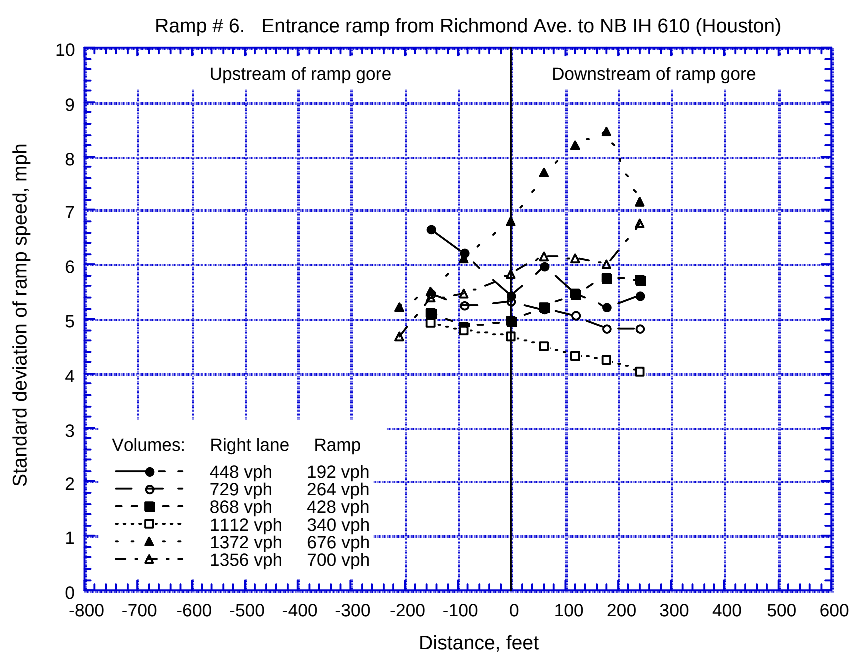

Figure A06 Standard Deviation of Ramp Speed, Ramp #6, Entrance Ramp from Richmond Avenue to NB IH 610, Houston

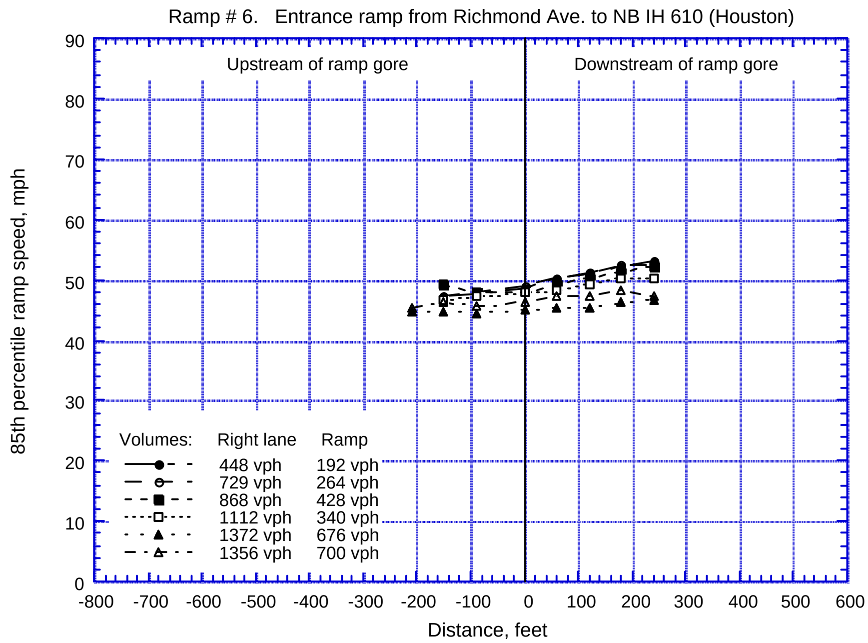

Figure A07 85th Percentile Ramp Speed, Ramp #6,
Entrance Ramp from Richmond Avenue to NB IH 610, Houston

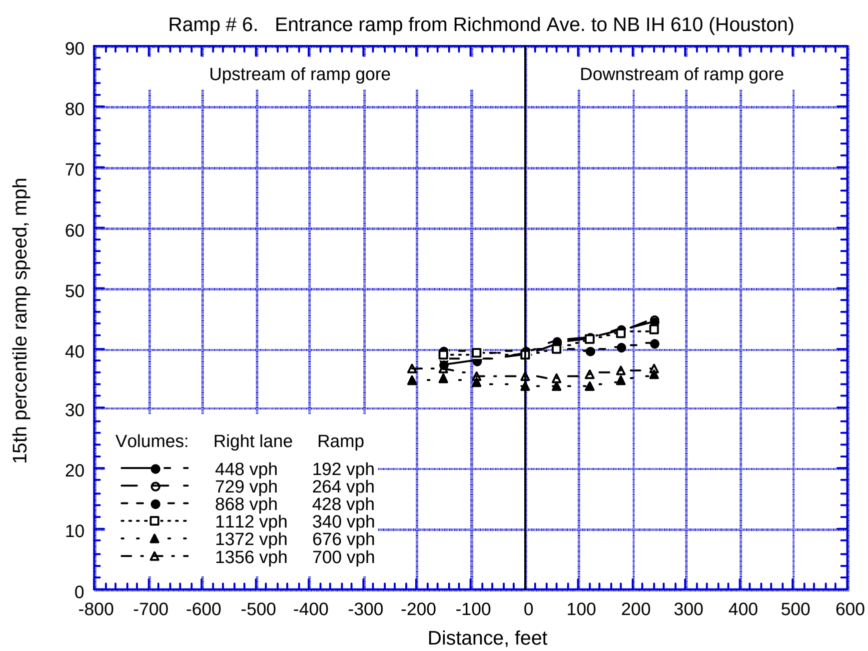

Figure A08 15th Percentile Ramp Speed, Ramp #6,
Entrance Ramp from Richmond Avenue to NB IH 610, Houston

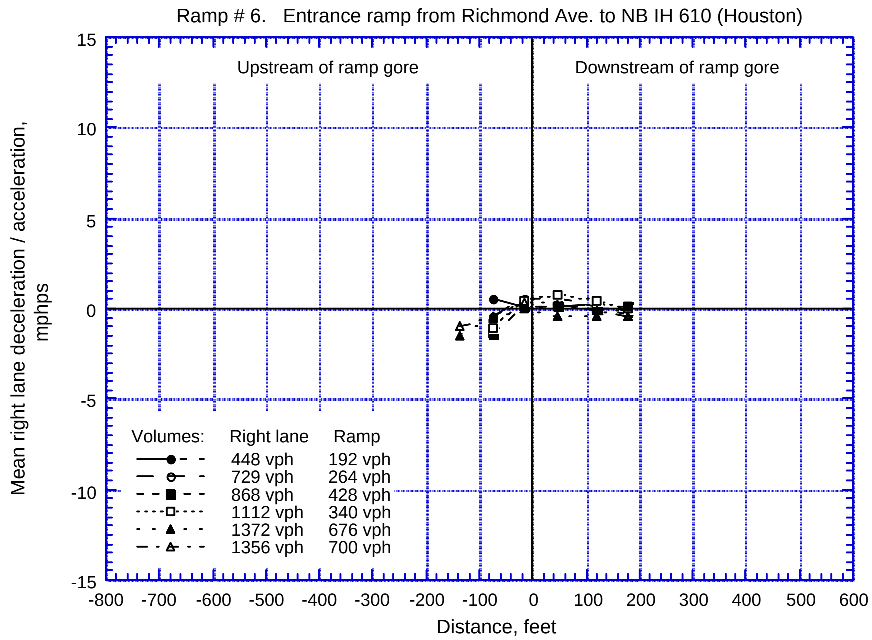

Figure A09 Mean Freeway Right Lane Acceleration/Deceleration, Ramp #6 Entrance Ramp from Richmond Avenue to NB IH 610, Houston

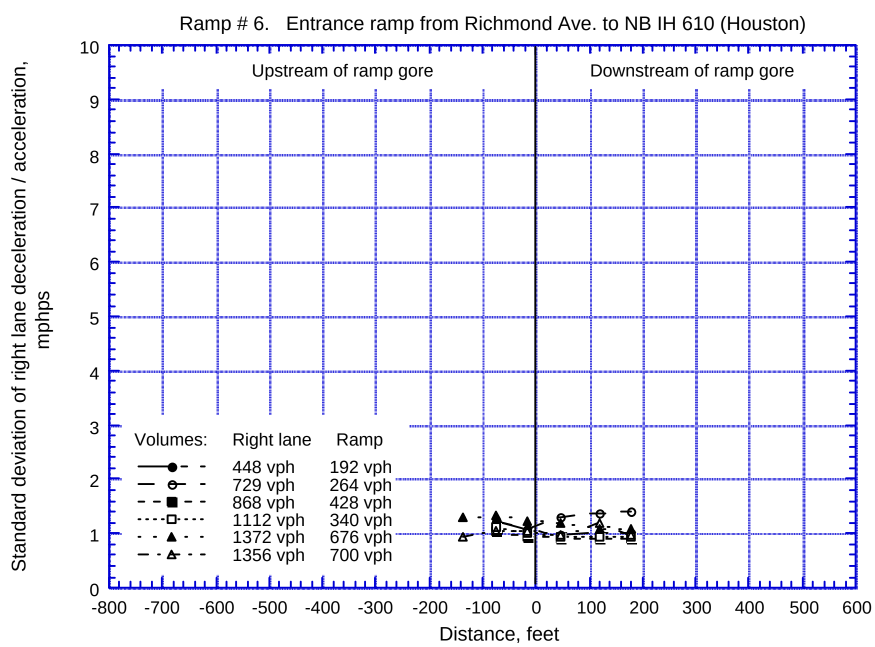

Figure A10 Standard Deviation of Freeway Right Lane Acceleration/Deceleration, Ramp #6, Entrance Ramp from Richmond Avenue to NB IH 610, Houston

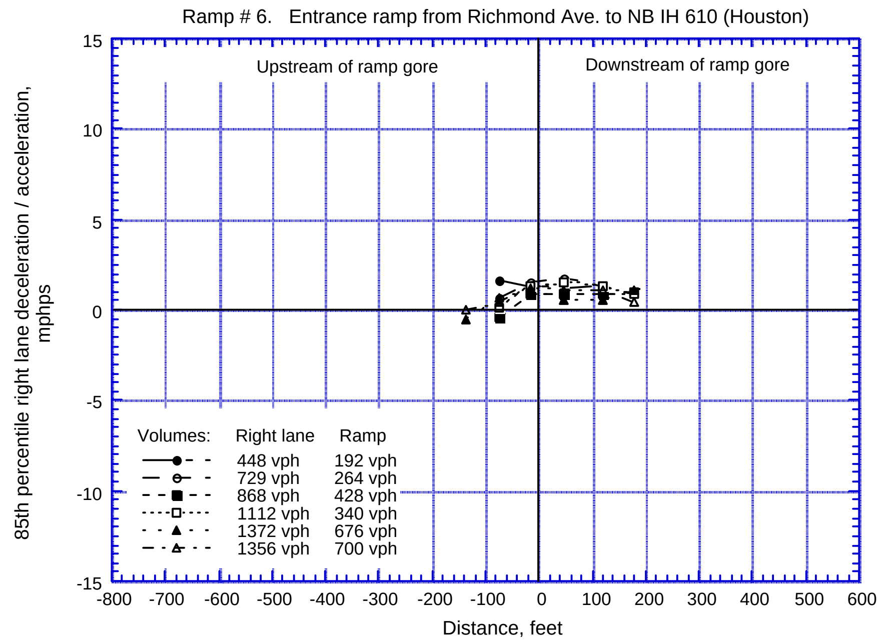

Figure A11 85th Percentile Freeway Right Lane Acceleration/Deceleration, Ramp #6, Entrance Ramp from Richmond Avenue to NB IH 610, Houston

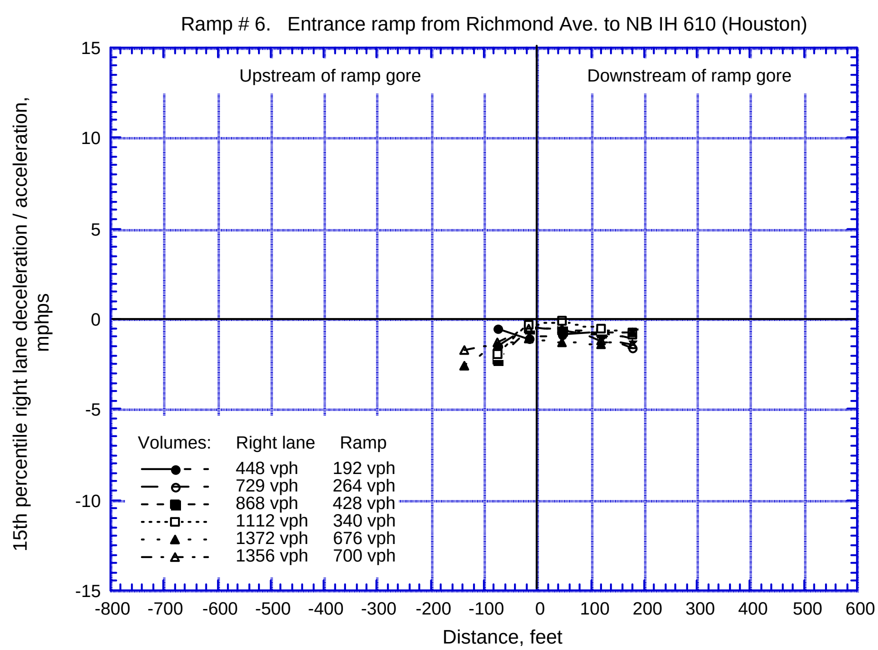

Figure A12 15th Percentile Freeway Right Lane Acceleration/Deceleration, Ramp #6, Entrance Ramp from Richmond Avenue to NB IH 610, Houston

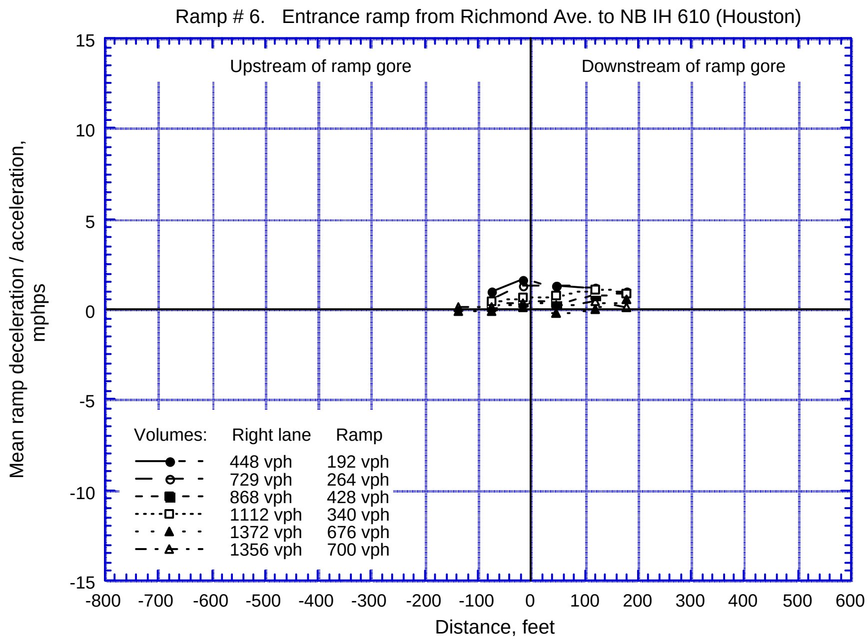

Figure A13 Mean Ramp Acceleration/Deceleration, Ramp #6, Entrance Ramp from Richmond Avenue to NB IH 610, Houston

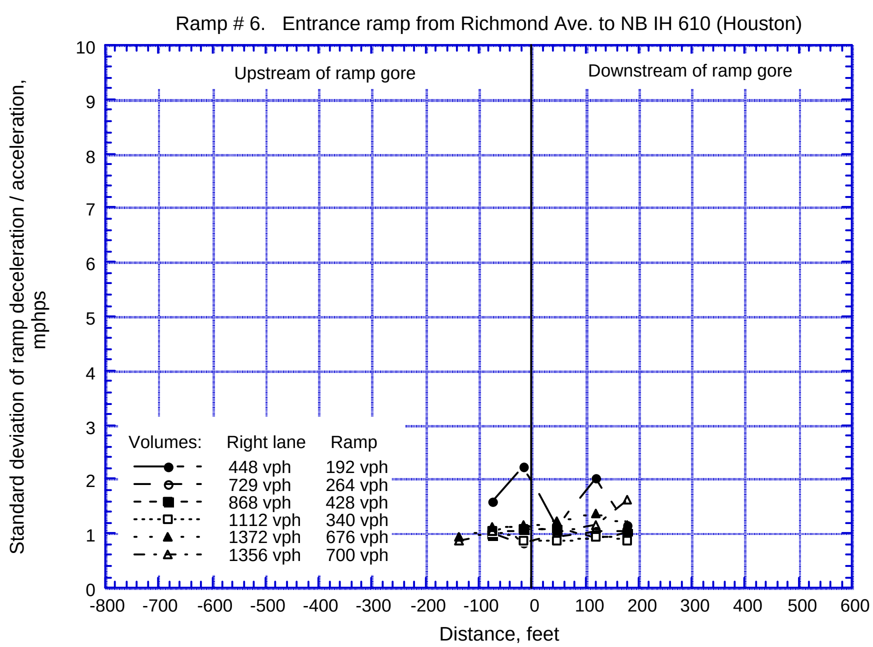

Figure A14 Standard Deviation of Ramp Acceleration/Deceleration, Ramp #6, Entrance Ramp from Richmond Avenue to NB IH 610, Houston

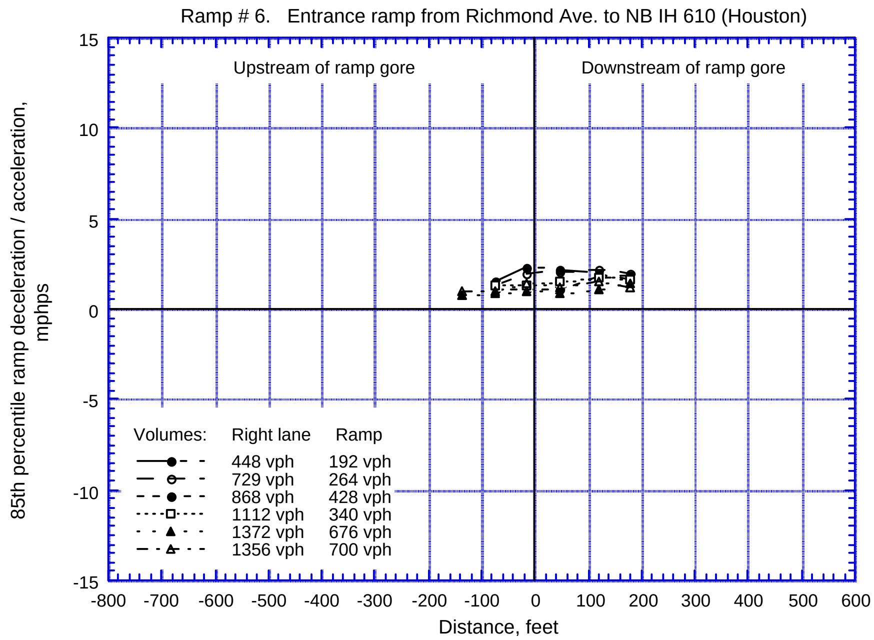

Figure A15 85th Percentile Ramp Acceleration/Deceleration, Ramp #6, Entrance Ramp from Richmond Avenue to NB IH 610, Houston

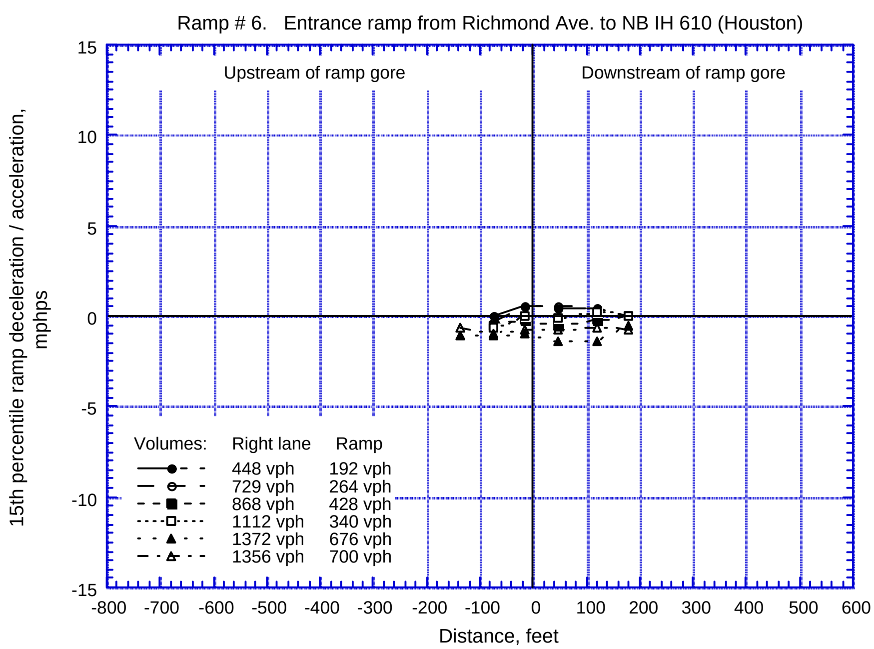

Figure A16 15th Percentile Ramp Acceleration/Deceleration, Ramp #6, Entrance Ramp from Richmond Avenue to NB IH 610, Houston

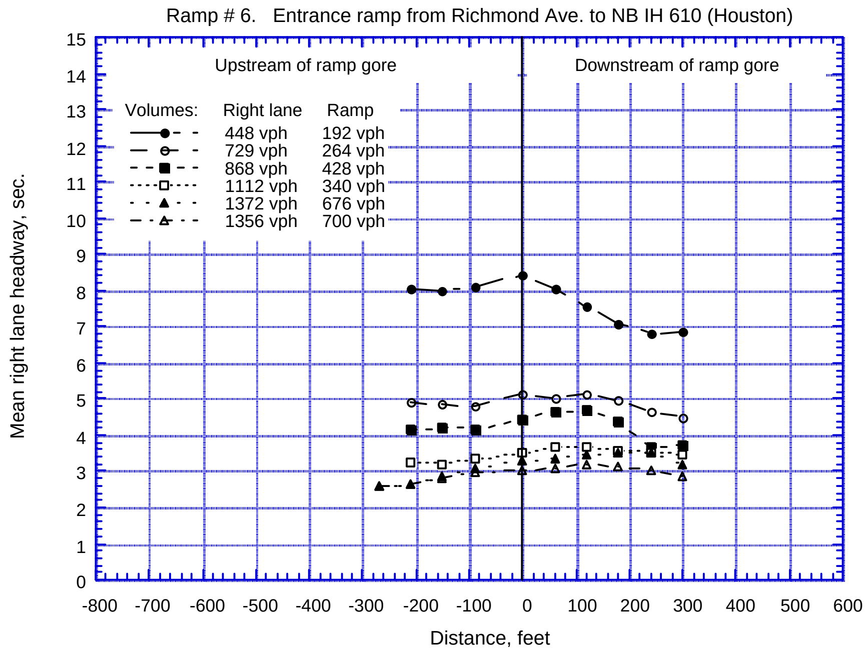

Figure A17 Mean Time Headway Freeway Right Lane, Ramp #6, Entrance Ramp from Richmond Avenue to NB IH 610, Houston

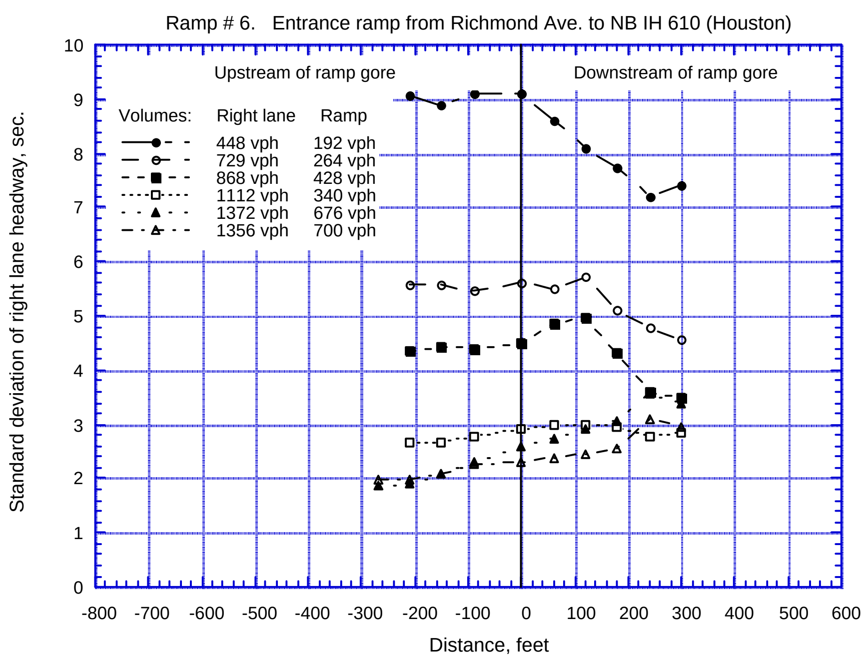

Figure A18 Standard Deviation of Time Headway Freeway Right Lane, Ramp #6, Entrance Ramp from Richmond Avenue to NB IH 610, Houston

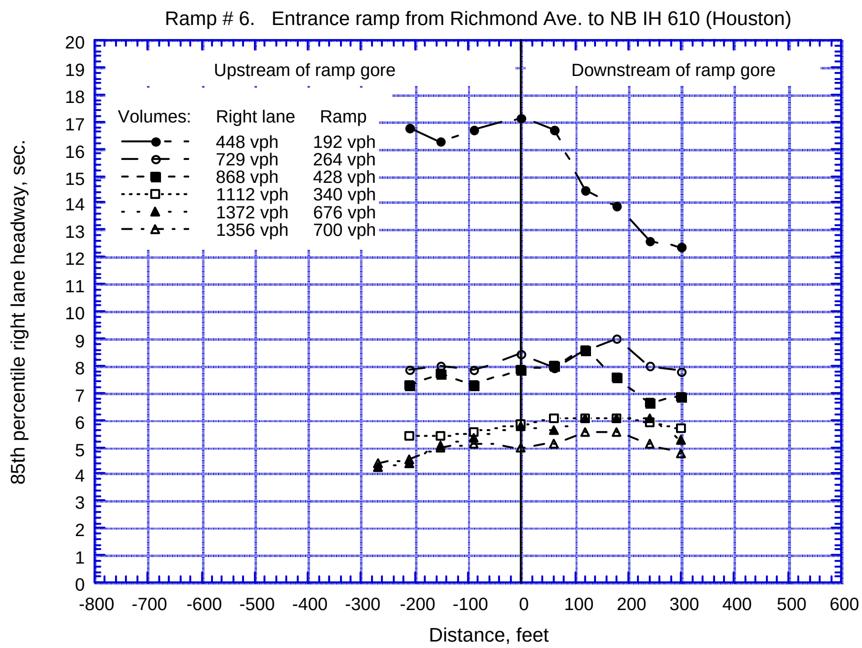

Figure A19 85th Percentile Time Headway Freeway Right Lane, Ramp #6, Entrance Ramp from Richmond Avenue to NB IH 610, Houston

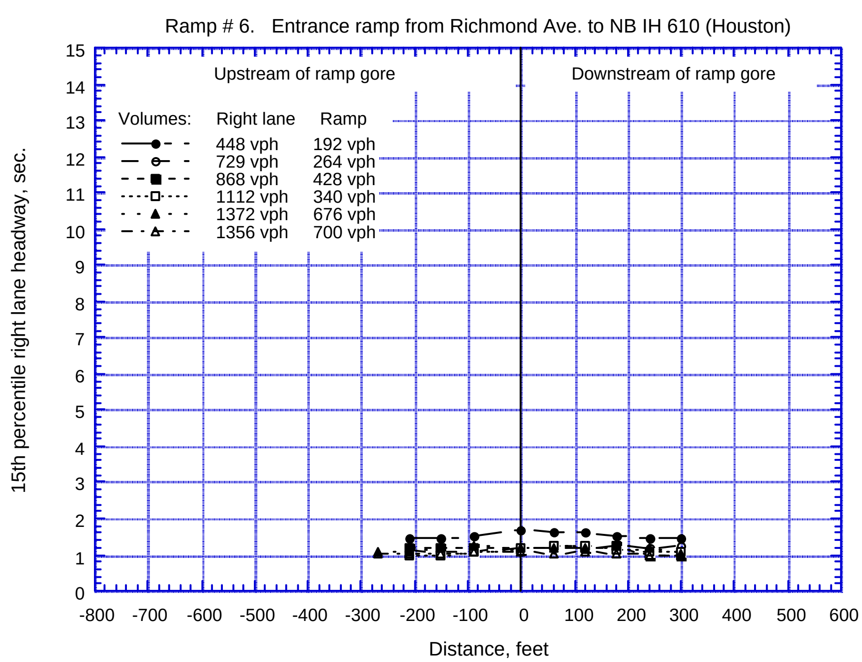

Figure A20 15th Percentile Time Headway Freeway Right Lane, Ramp #6, Entrance Ramp from Richmond Avenue to NB IH 610, Houston

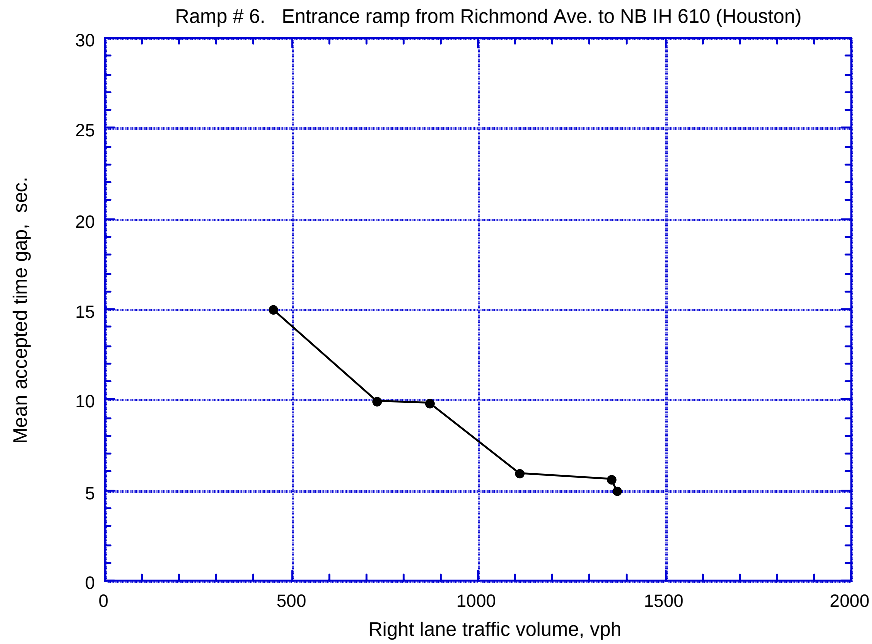

Figure A21 Mean Freeway Time Gap Accepted by Ramp Vehicles Versus Right Freeway Lane Traffic Volume, Ramp #6, Entrance Ramp from Richmond Avenue to NB IH 610, Houston

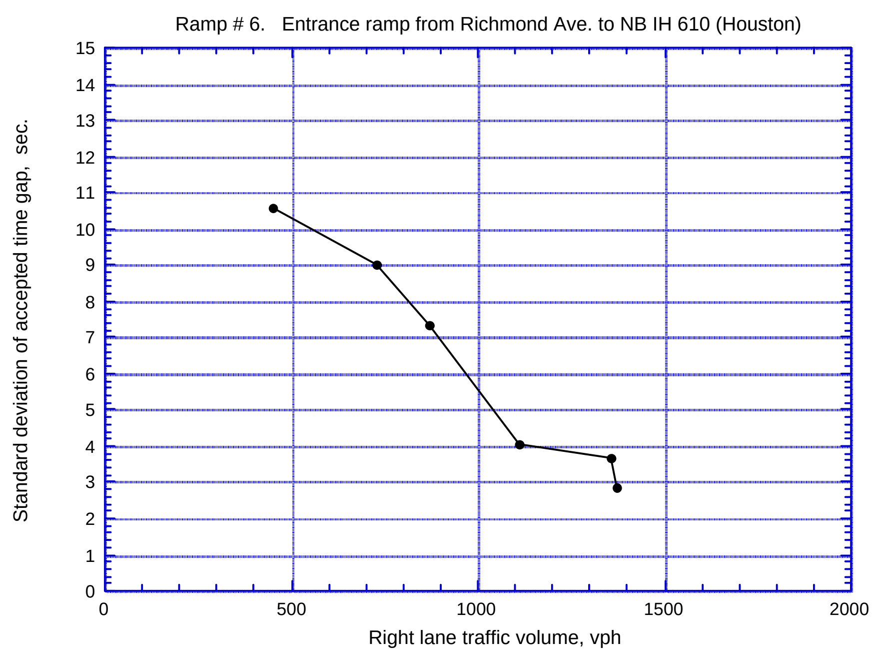

Figure A22 Standard Deviation of Freeway Time Gap Accepted by Ramp Vehicles, Ramp #6, Entrance Ramp from Richmond Avenue to NB IH 610, Houston

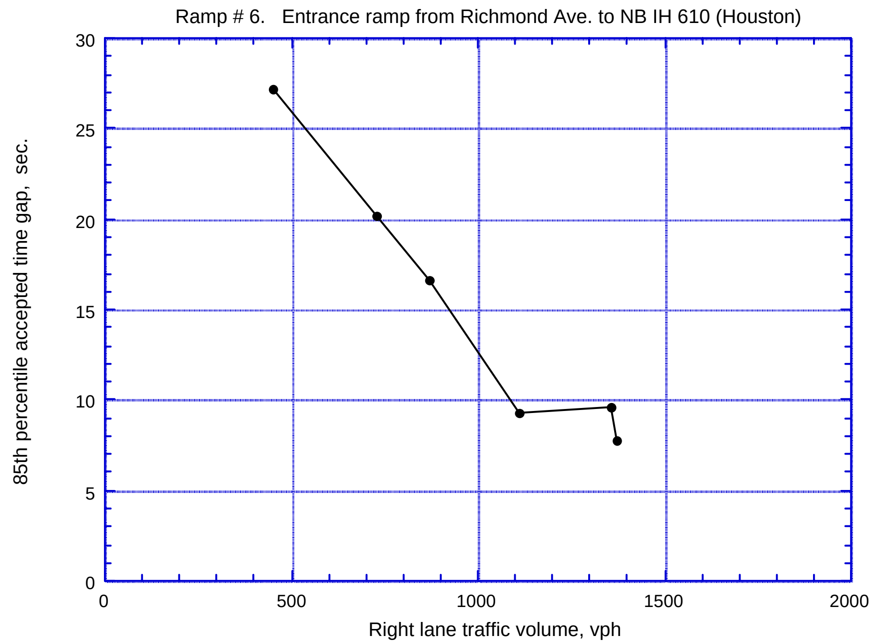

Figure A23 85th Percentile Freeway Time Gap Accepted by Ramp Vehicles, Ramp #6, Entrance Ramp from Richmond Avenue to NB IH 610, Houston

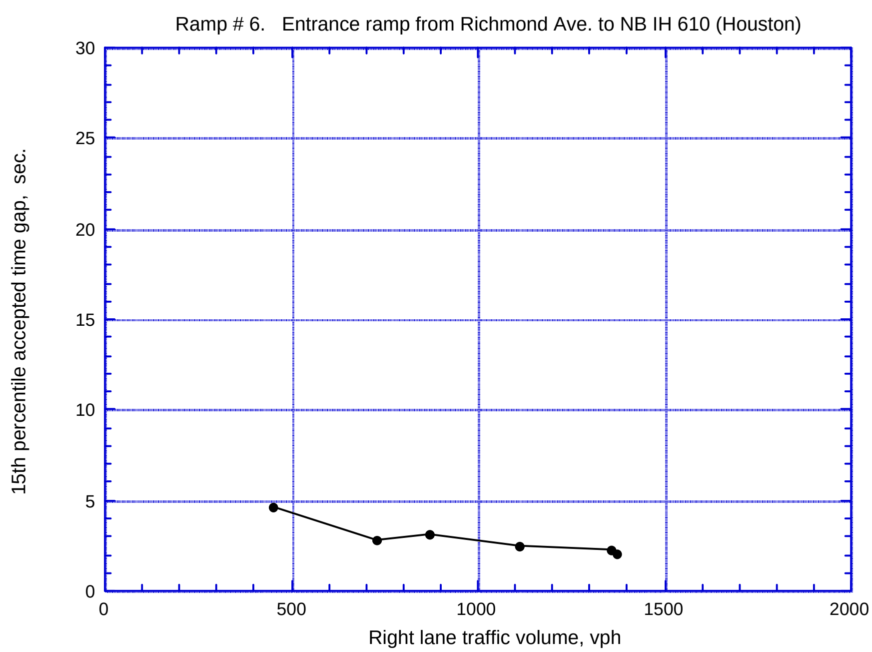

Figure A24 15th Percentile Freeway Time Gap Accepted by Ramp Vehicles, Ramp #6, Entrance Ramp from Richmond Avenue to NB IH 610, Houston

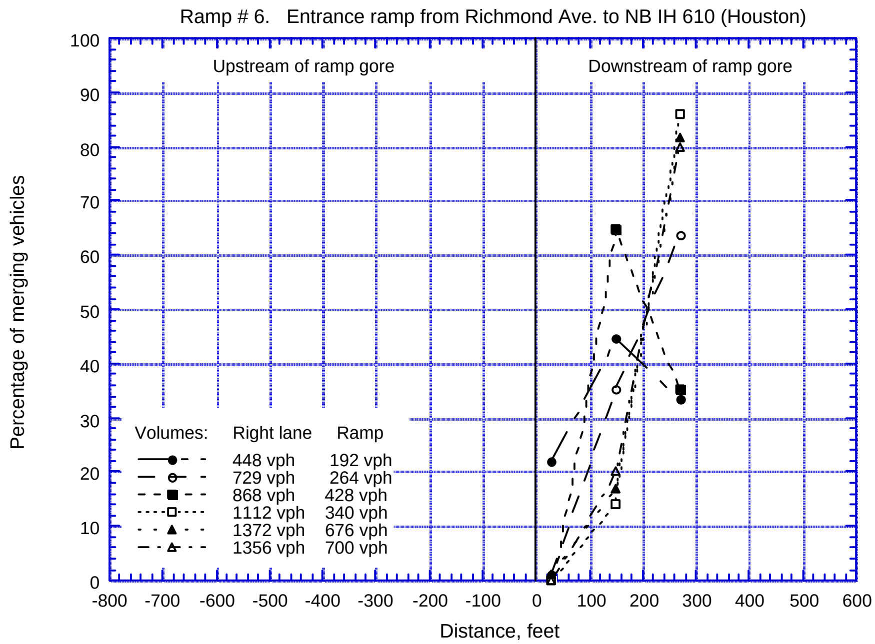

Figure A25 Ramp Vehicle Merging Location Percentage, Ramp #6, Entrance Ramp from Richmond Avenue to NB IH 610, Houston

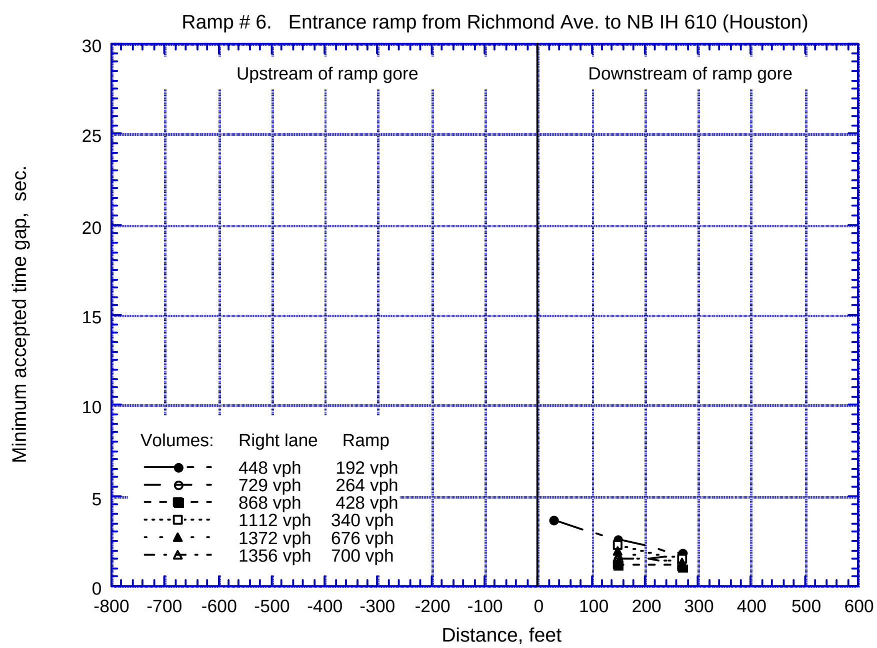

Figure A26 Minimum Time Gap Accepted by Ramp Vehicles, Ramp #6, Entrance Ramp from Richmond Avenue to NB IH 610, Houston

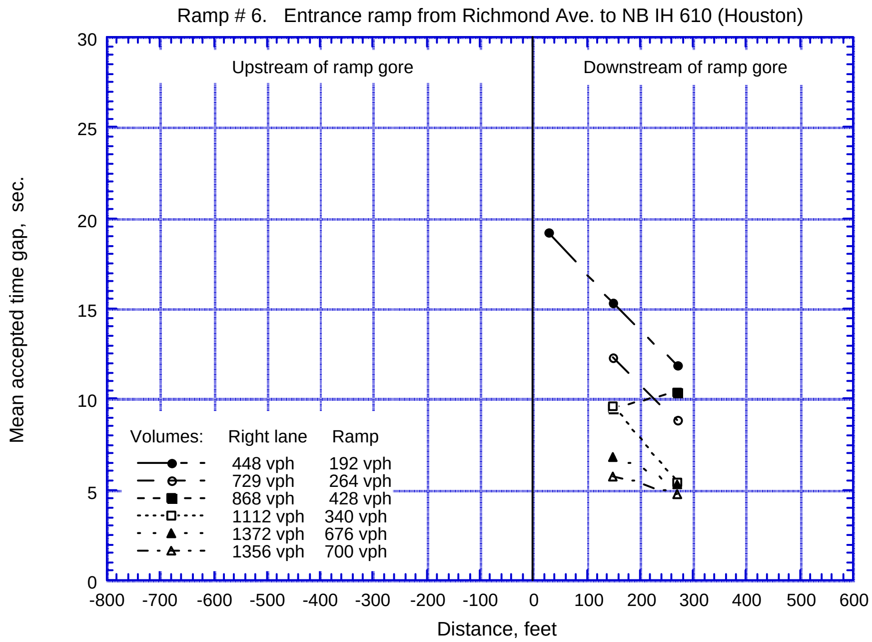

Figure A27 Mean Freeway Time Gap Accepted by Ramp Vehicles Versus Merging Location, Ramp #6, Entrance Ramp from Richmond Avenue to NB IH 610, Houston

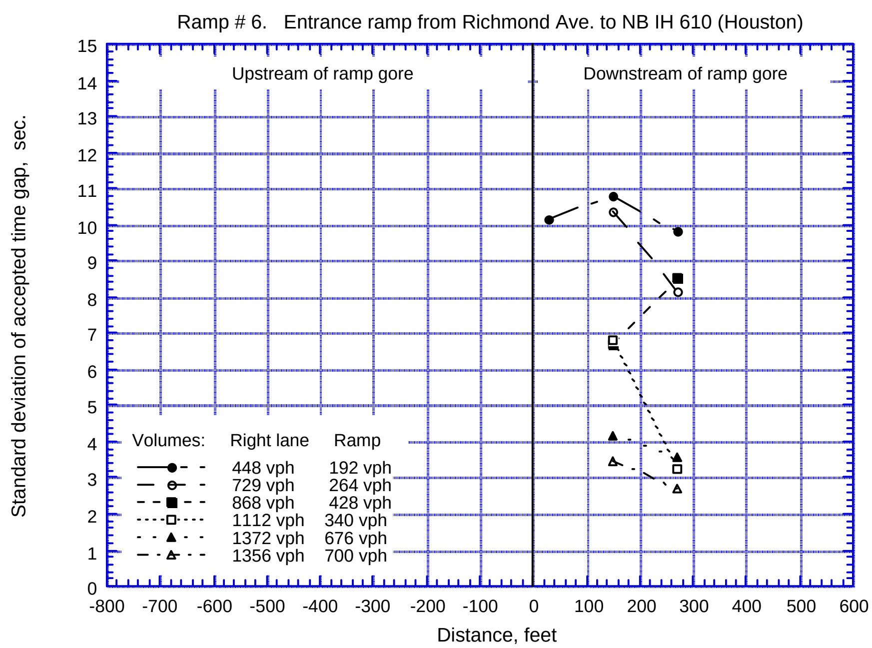

Figure A28 Standard Deviation of Freeway Time Gap Accepted by Ramp Vehicles Versus Merging Location, Ramp #6, Entrance Ramp from Richmond Avenue to NB IH 610, Houston

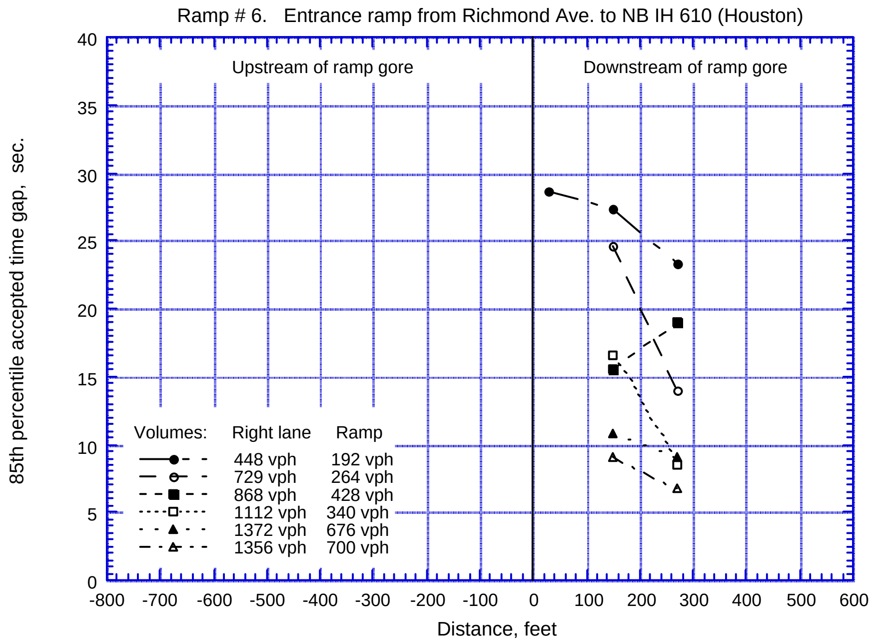

Figure A29 85th Percentile Freeway Time Gap Accepted by Ramp Vehicles Versus Merging Location, Ramp #6, Entrance Ramp from Richmond Avenue to NB IH 610, Houston

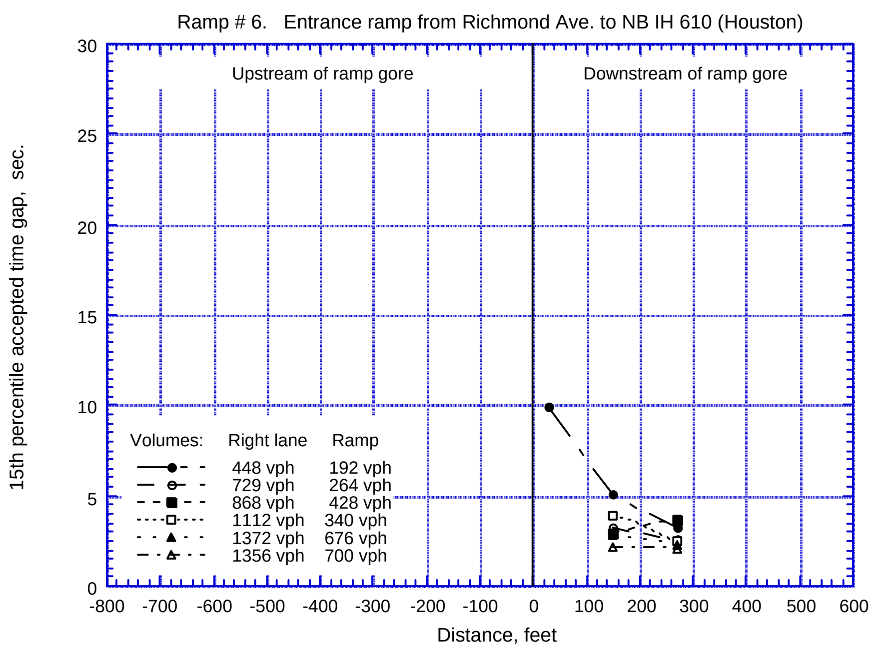

Figure A30 15th Percentile Freeway Time Gap Accepted by Ramp Vehicles Versus Merging Location, Ramp #6, Entrance Ramp from Richmond Avenue to NB IH 610, Houston

APPENDIX B

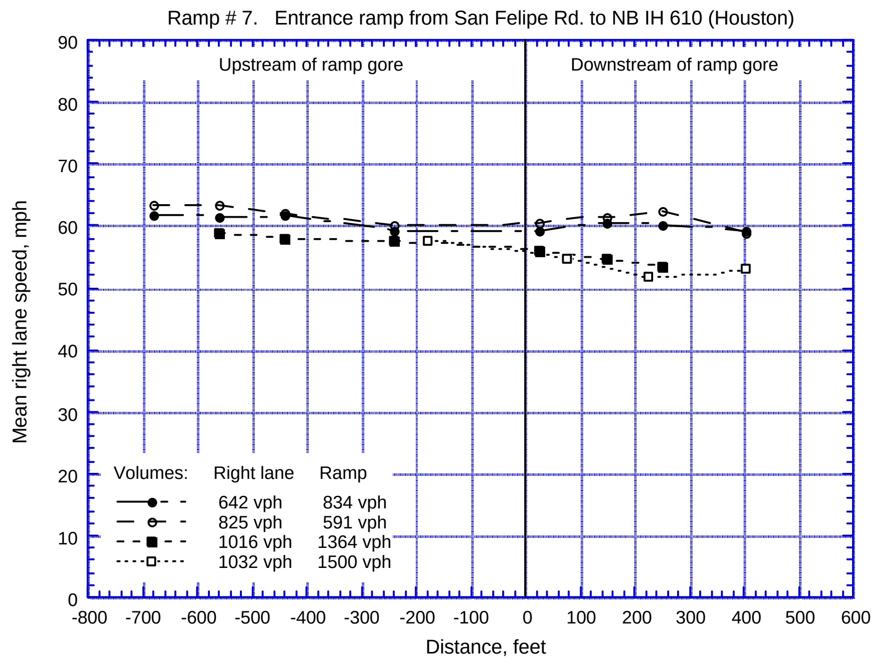

Figure B01 Mean Freeway Right Lane Speed, Ramp # 7,
Entrance Ramp from San Felipe Rd. to NB IH 610, Houston

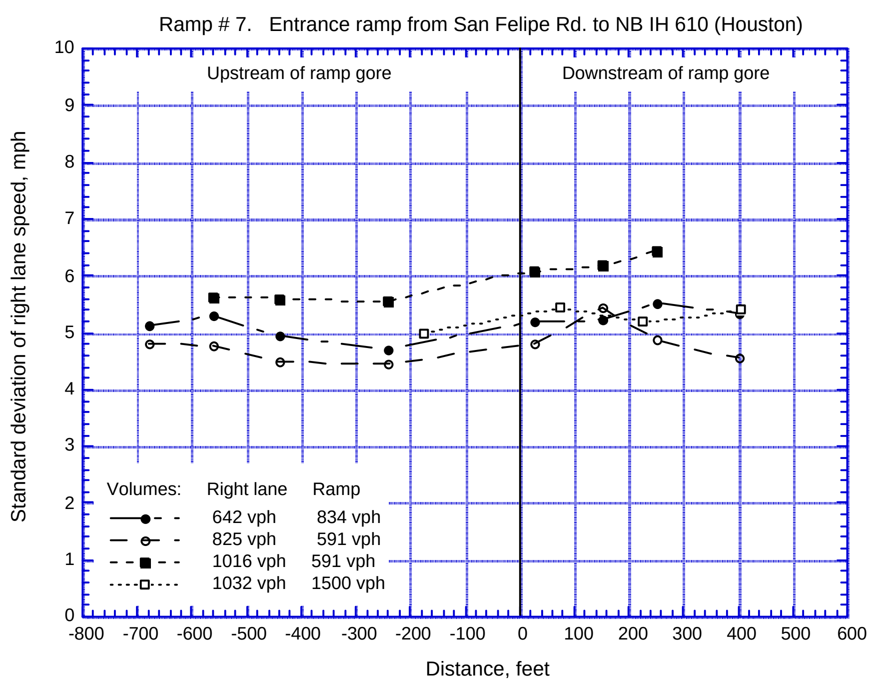

Figure B02 Standard Deviation of Freeway Right Lane Speed, Ramp # 7,
Entrance Ramp from San Felipe Rd. to NB IH 610, Houston

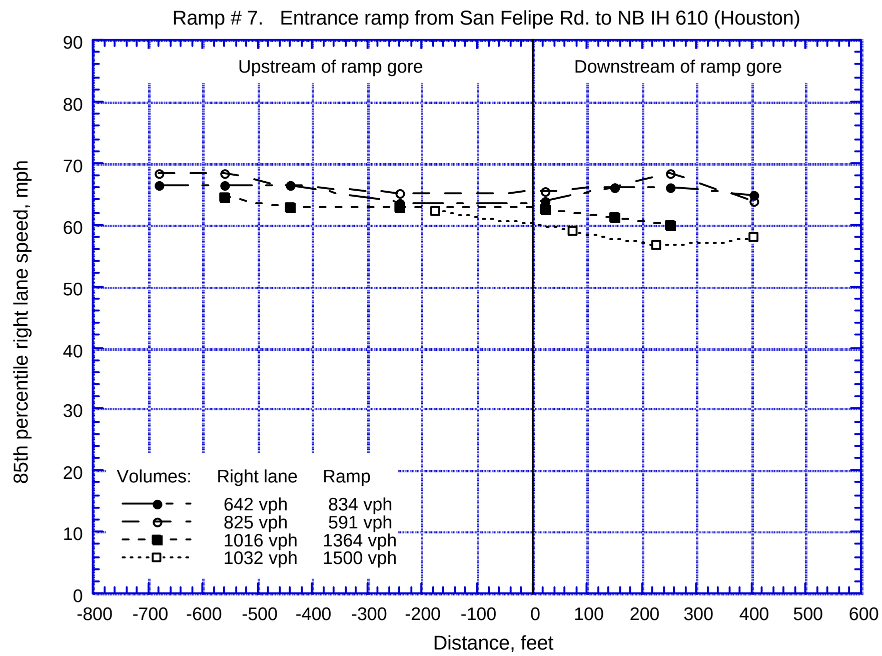

Figure B03 85th Percentile Freeway Right Lane Speed, Ramp # 7,
Entrance Ramp from San Felipe Rd. to NB IH 610, Houston

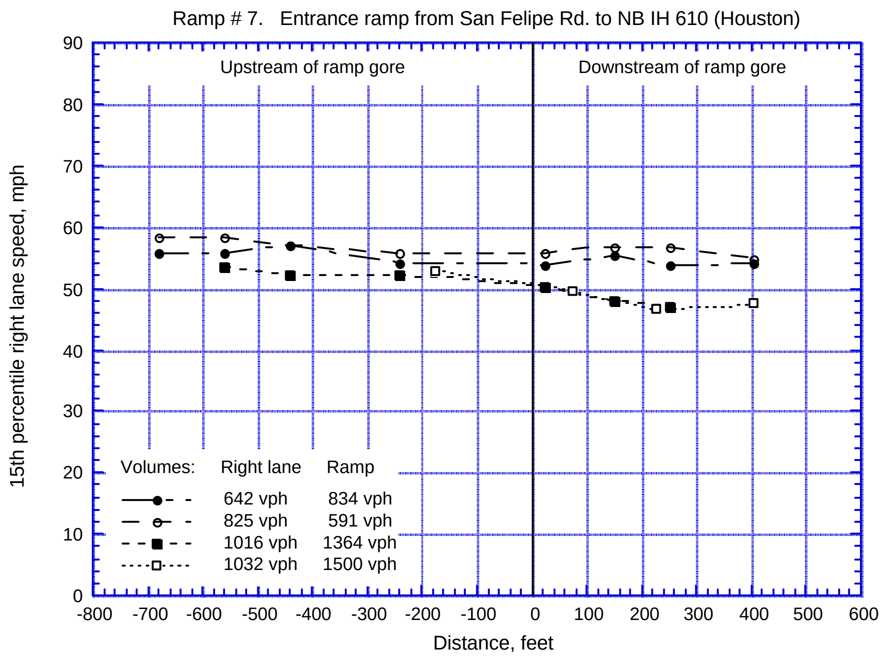

Figure B04 15th Percentile Freeway Right Lane Speed, Ramp # 7,
Entrance Ramp from San Felipe Rd. to NB IH 610, Houston

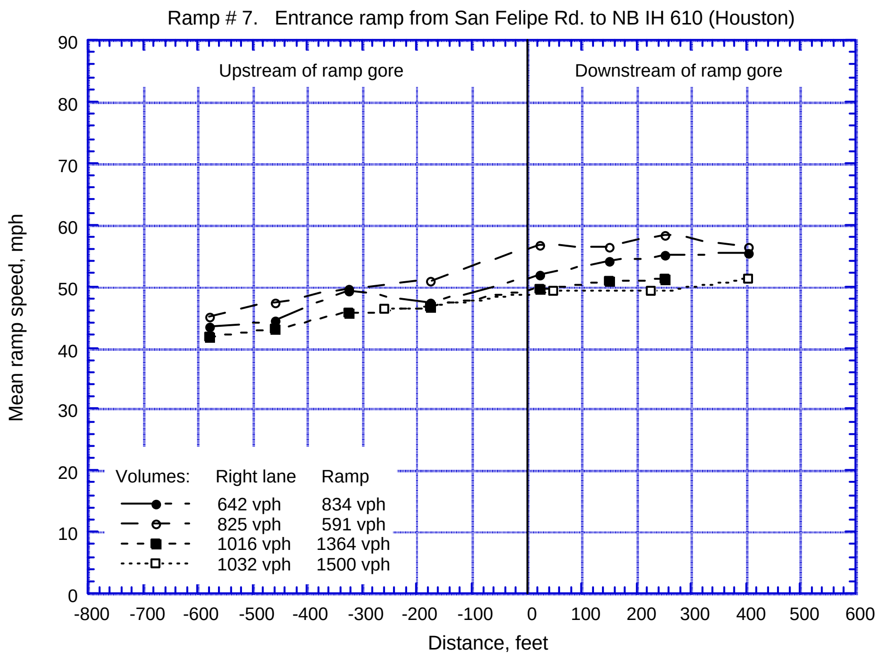

Figure B05 Mean Ramp Speed, Ramp # 7, Entrance Ramp from San Felipe Rd. to NB IH 610, Houston

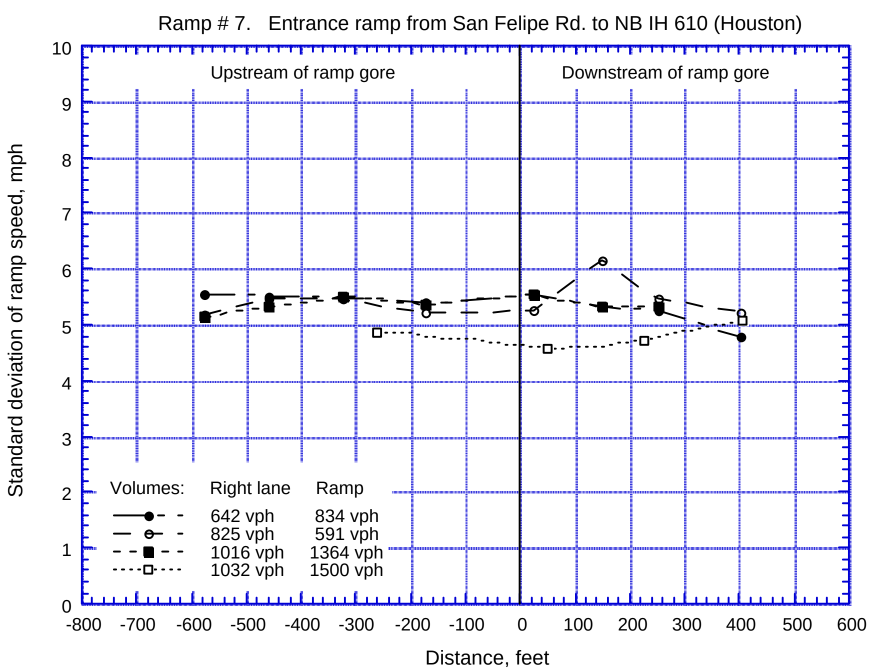

Figure B06 Standard Deviation of Ramp Speed, Ramp # 7, Entrance Ramp from San Felipe Rd. to NB IH 610, Houston

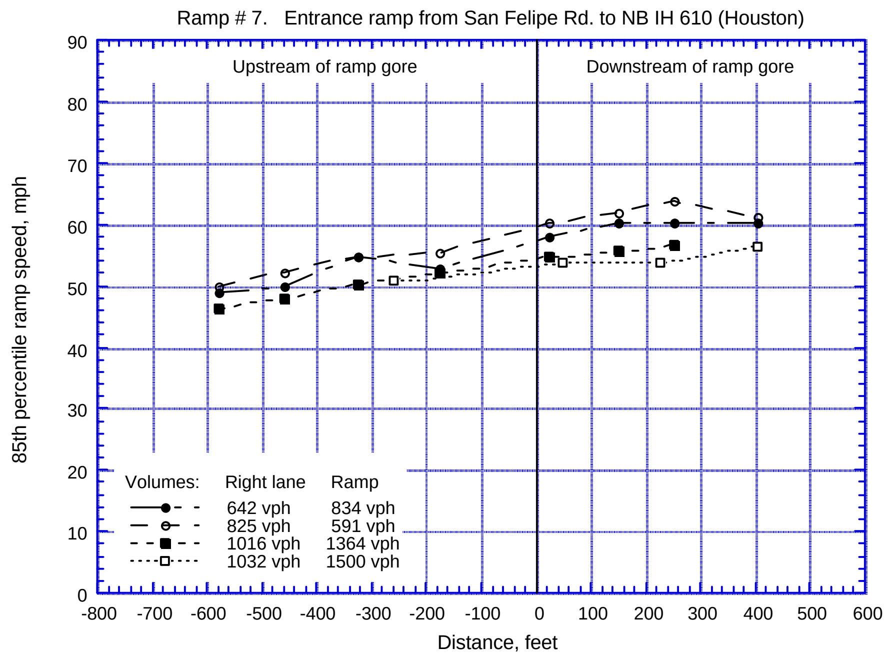

Figure B07 85th Percentile Ramp Speed, Ramp # 7,
Entrance Ramp from San Felipe Rd. to NB IH 610, Houston

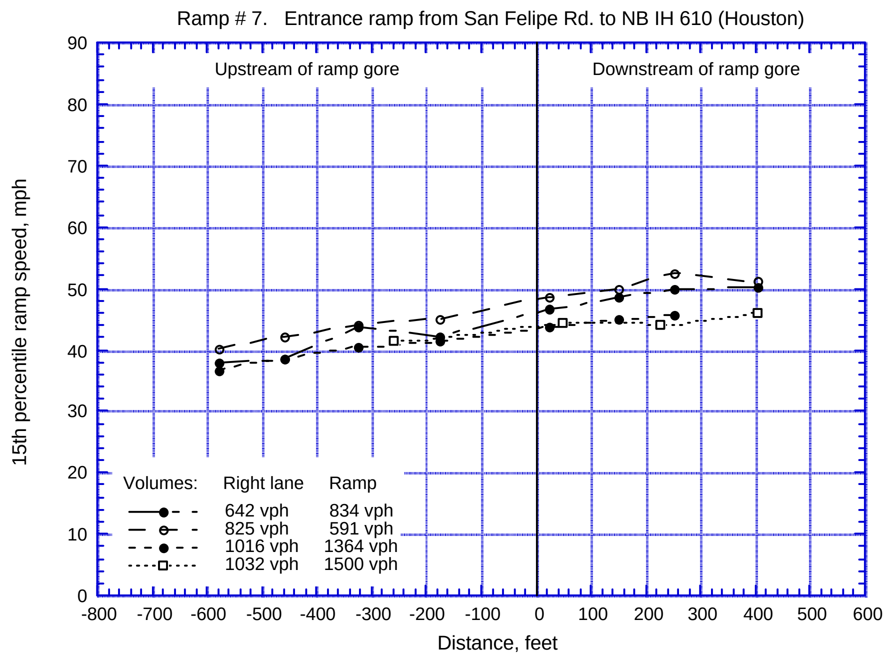

Figure B08 15th Percentile Ramp Speed, Ramp # 7,
Entrance Ramp from San Felipe Rd. to NB IH610, Houston

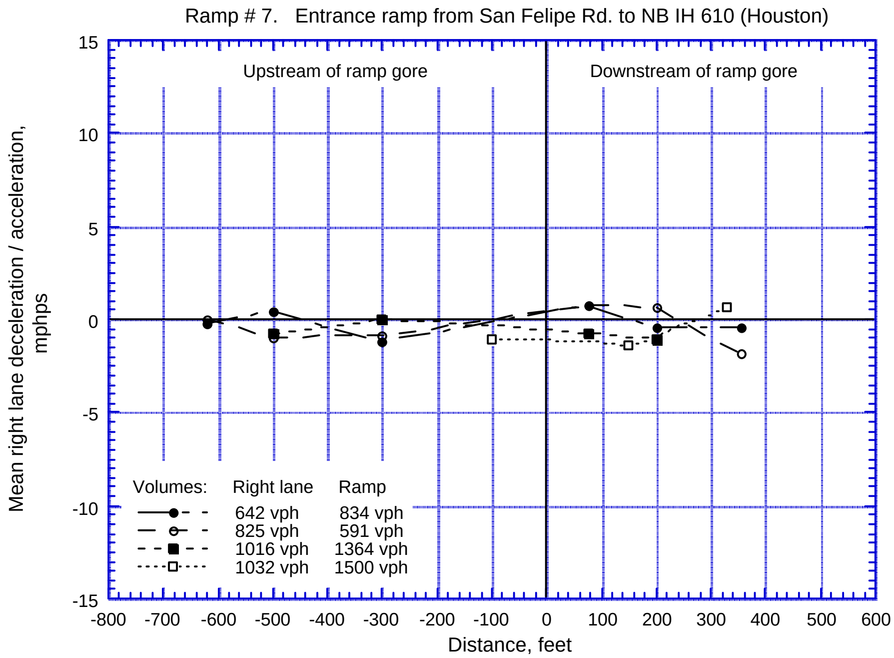

Figure B09 Mean Freeway Right Lane Acceleration/Deceleration, Ramp # 7, Entrance Ramp from San Felipe Rd. to NB IH 610, Houston

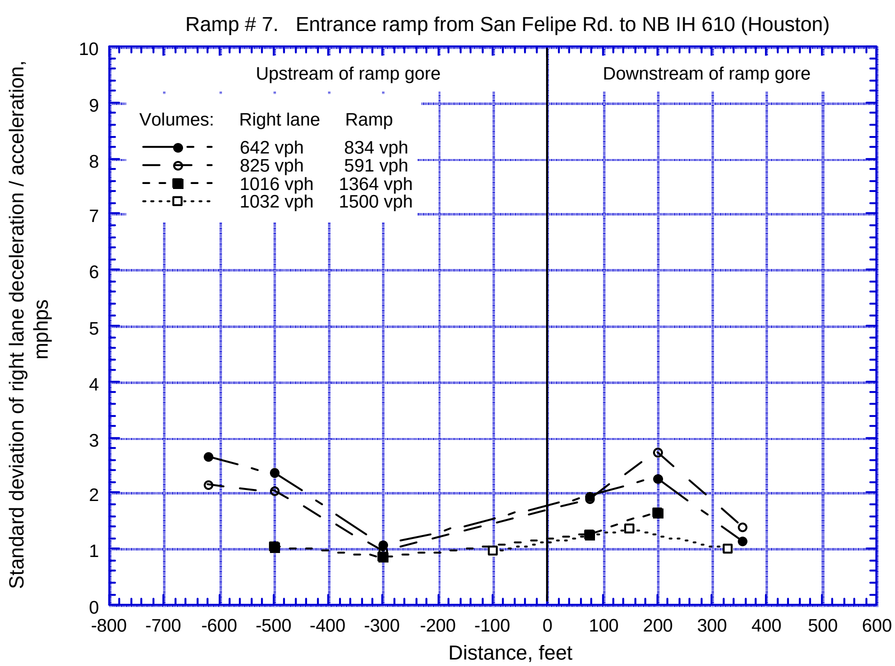

Figure B10 Standard Deviation Of Freeway Right Lane Acceleration/Deceleration, Ramp # 7, Entrance Ramp from San Felipe Rd. to NB IH 610, Houston

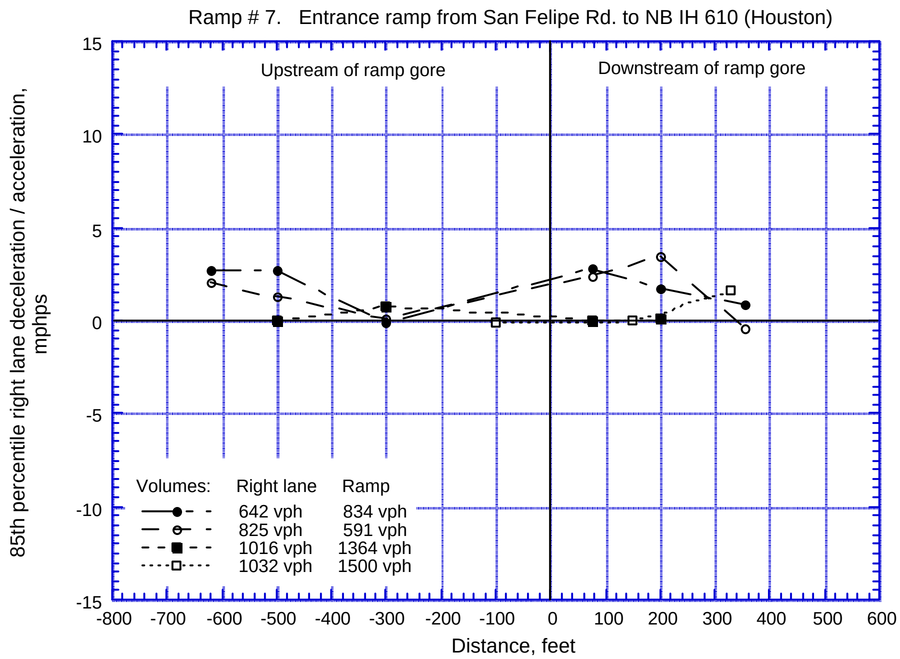

Figure B11 85th Percentile Freeway Right Lane Acceleration/Deceleration, Ramp # 7, Entrance Ramp from San Felipe Rd. to NB IH 610, Houston

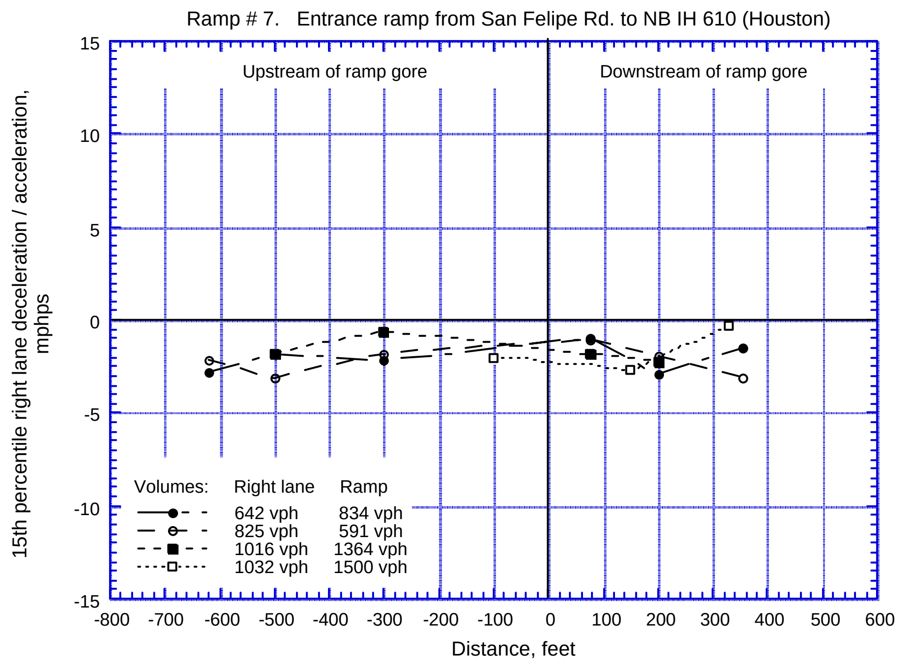

Figure B12 15th Percentile Freeway Right Lane Acceleration/Deceleration, Ramp # 7, Entrance Ramp from San Felipe Rd. to NB IH 610, Houston

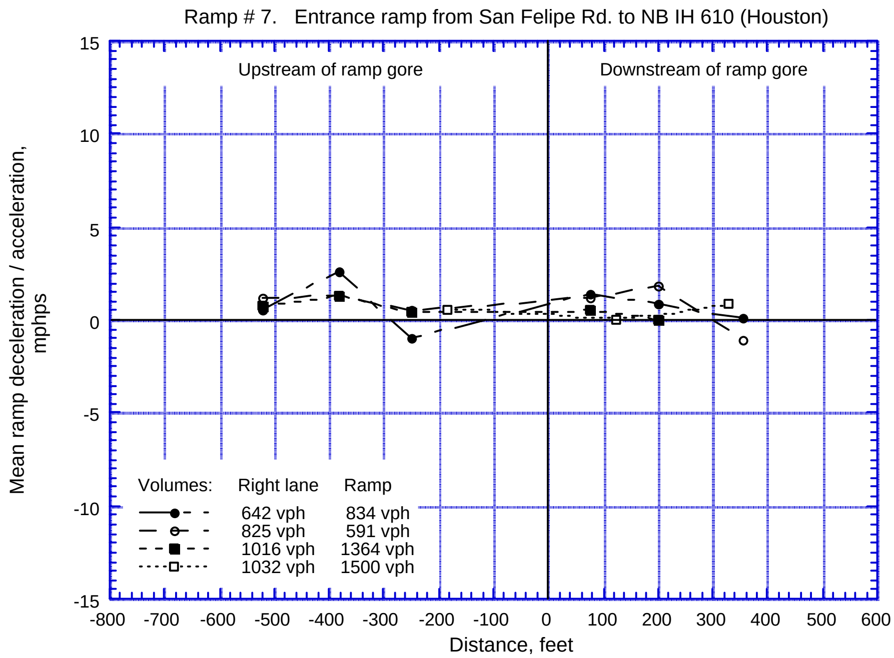

Figure B13 Mean Ramp Acceleration/Deceleration, Ramp # 7, Entrance Ramp from San Felipe Rd. to NB IH 610, Houston

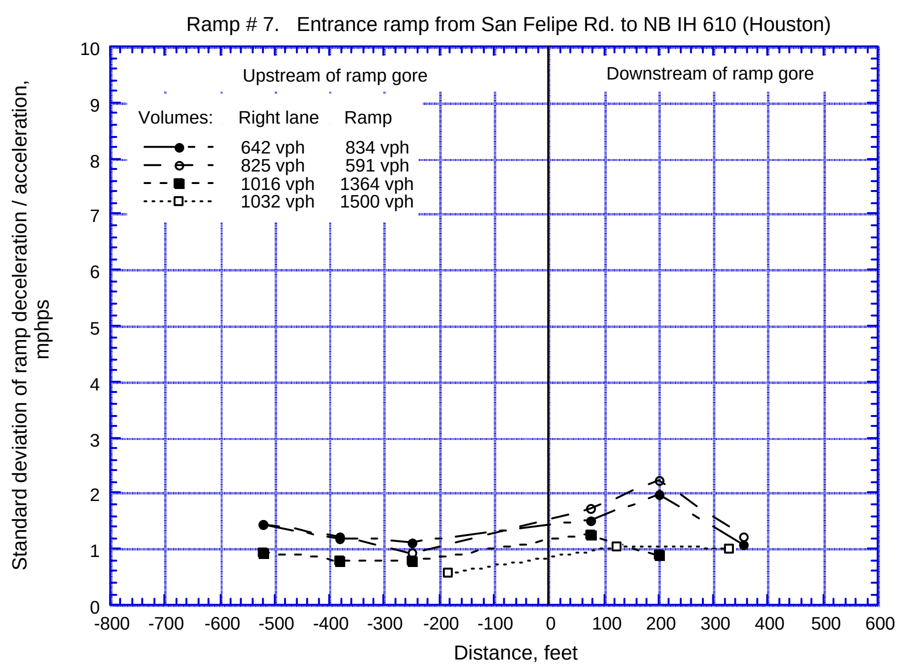

Figure B14 Standard Deviation of Ramp Acceleration/Deceleration, Ramp # 7, Entrance Ramp from San Felipe Rd. to NB IH 610, Houston

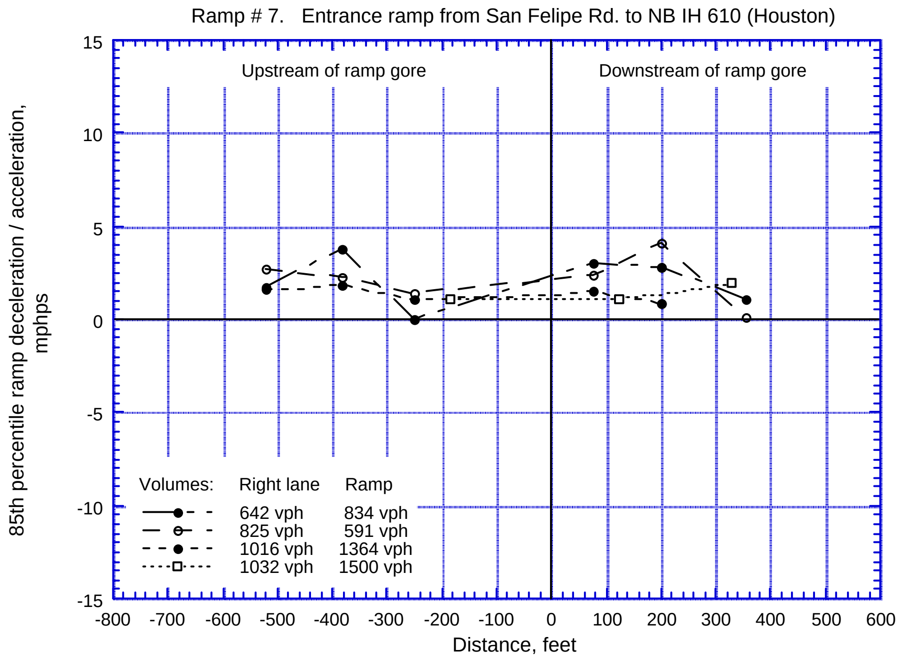

Figure B15 85th Percentile Ramp Acceleration/Deceleration, Ramp # 7, Entrance Ramp from San Felipe Rd. to NB IH 610, Houston

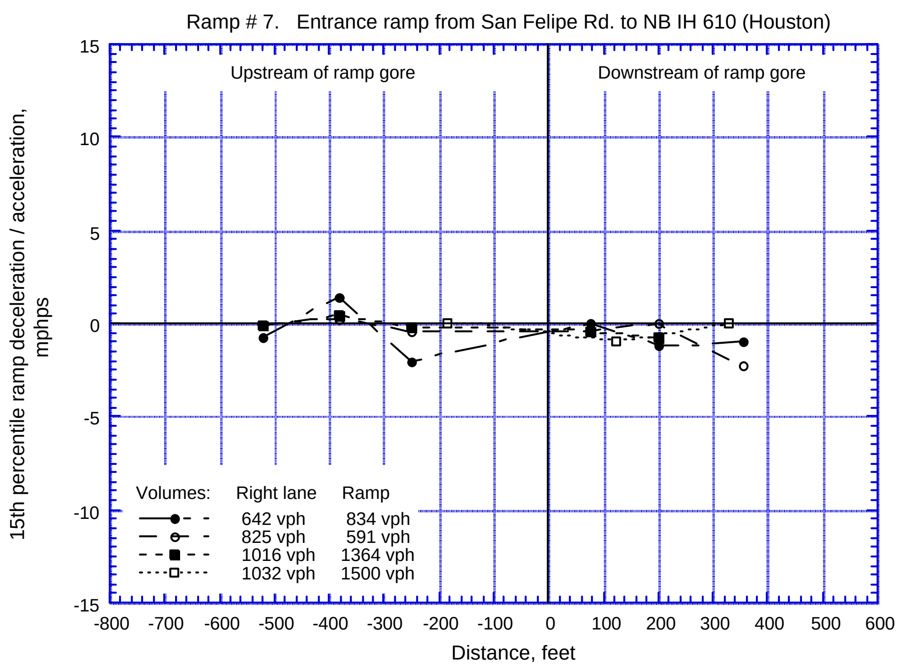

Figure B16 15th Percentile Ramp Acceleration/Deceleration, Ramp # 7, Entrance Ramp from San Felipe Rd. to NB IH 610, Houston

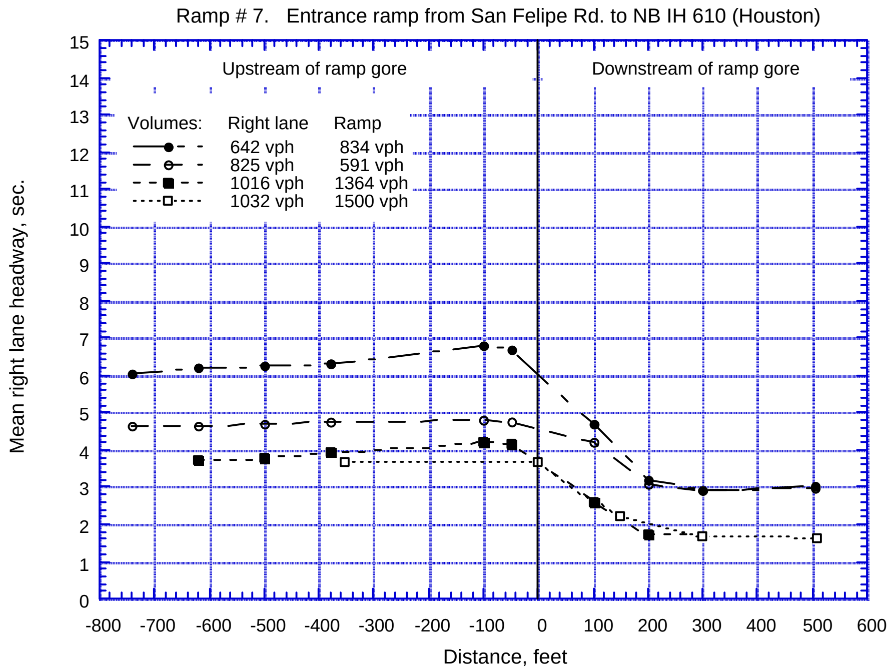

Figure B17 Mean Time Headway Freeway Right Lane, Ramp # 7, Entrance Ramp from San Felipe Rd. to NB IH 610, Houston

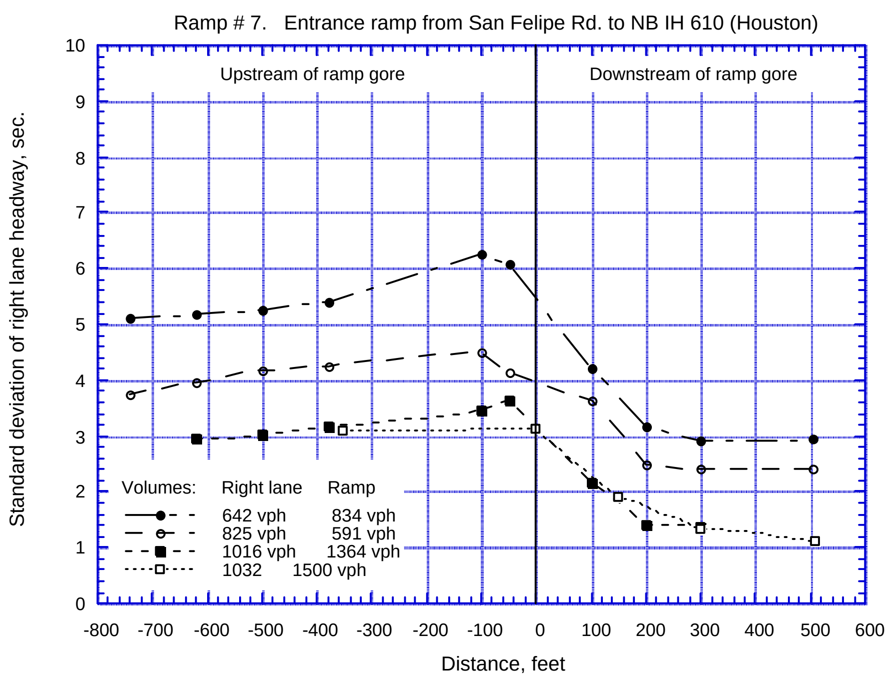

Figure B18 Standard Deviation of Time Headway Freeway Right Lane, Ramp # 7, Entrance Ramp from San Felipe Rd. to NB IH 610, Houston

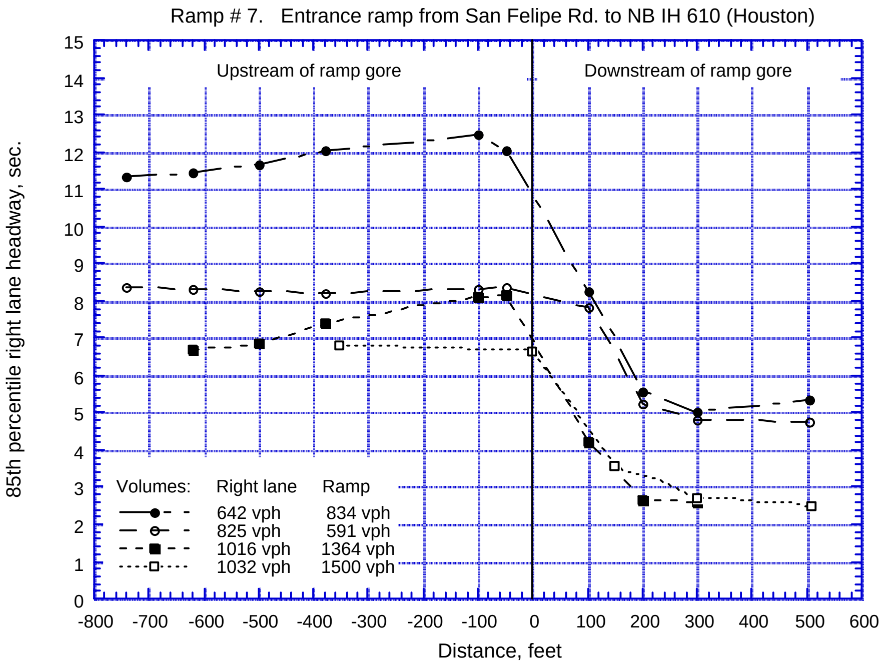

Figure B19 85th Percentile Time Headway Freeway Right Lane, Ramp # 7, Entrance Ramp from San Felipe Rd. to NB IH 610, Houston

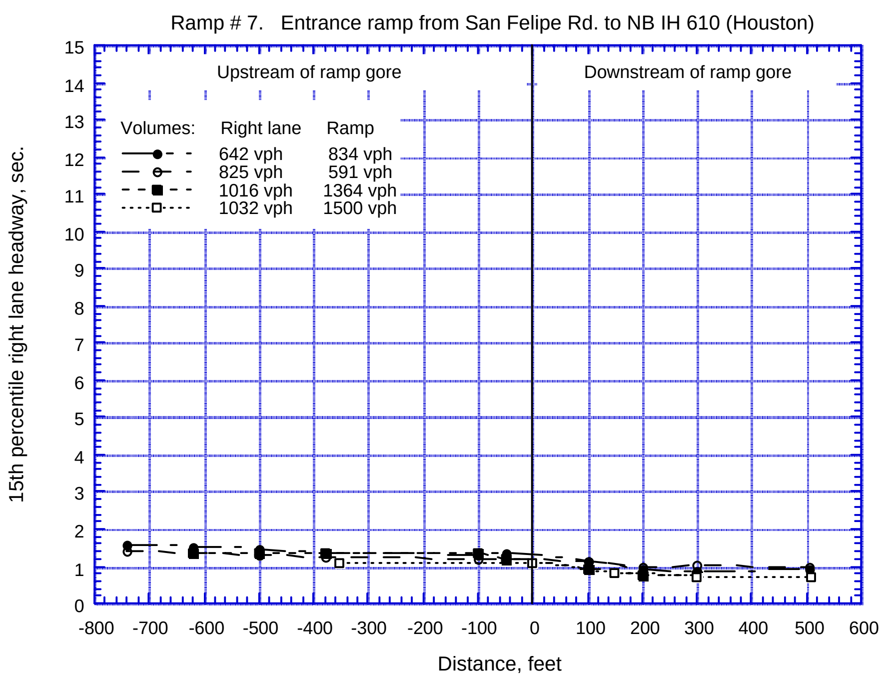

Figure B20 15th Percentile Time Headway Freeway Right Lane, Ramp # 7, Entrance Ramp from San Felipe Rd. to NB IH 610, Houston

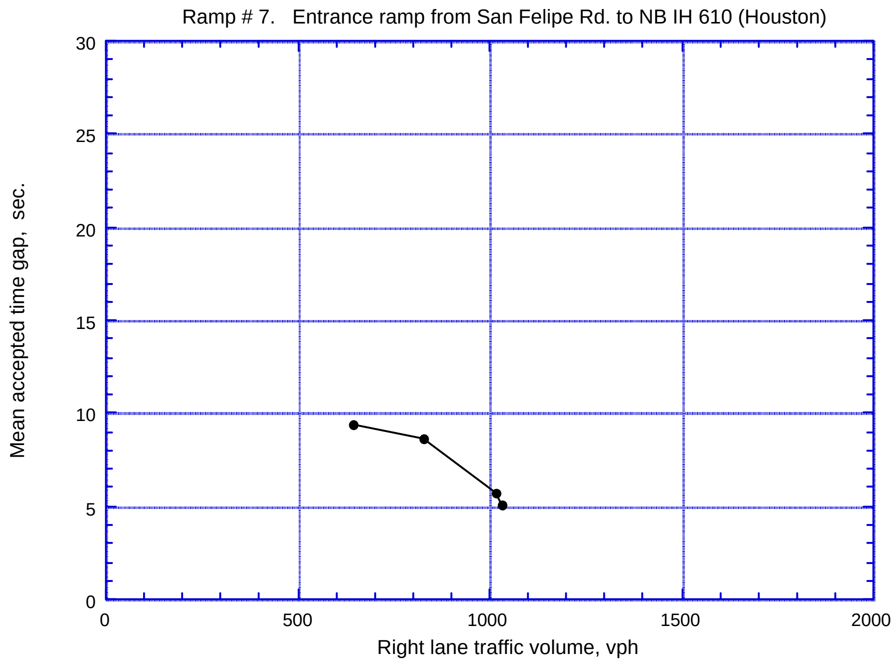

Figure B21 Mean Freeway Time Gap Accepted by Ramp Vehicles Versus Right Freeway Lane Traffic Volume, Ramp # 7, Entrance Ramp from San Felipe Rd. to NB IH 610, Houston

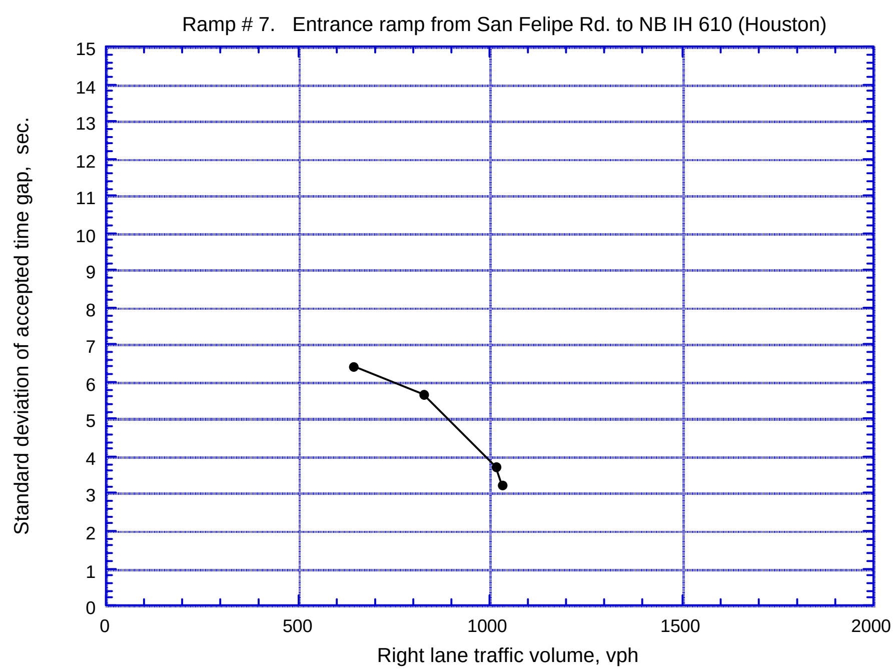

Figure B22 Standard Deviation of Freeway Time Gap Accepted by Ramp Vehicles, Ramp # 7, Entrance Ramp from San Felipe Rd. to NB IH 610, Houston

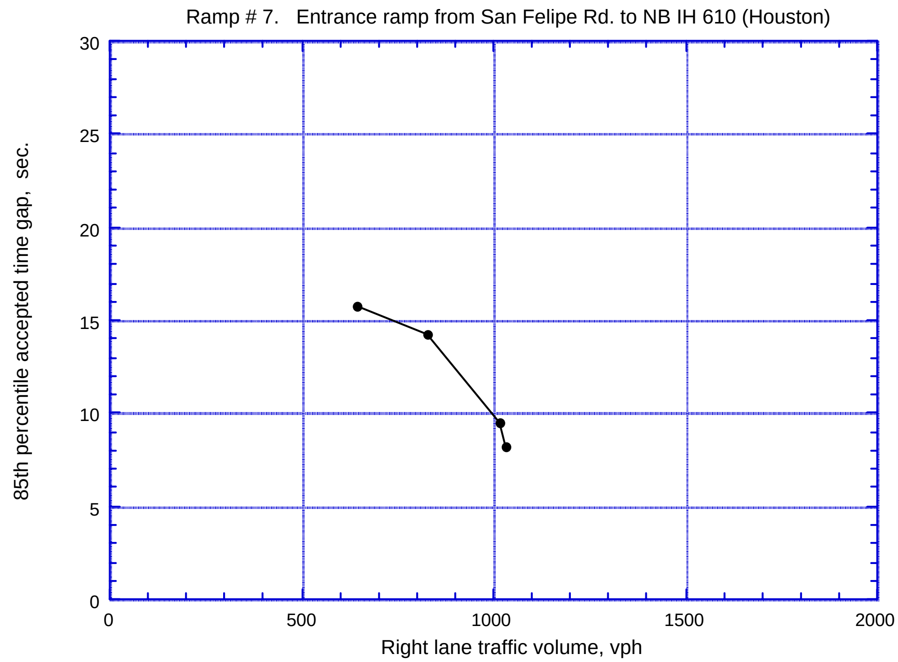

Figure B23 85th Percentile Freeway Time Gap Accepted by Ramp Vehicles, Ramp # 7, Entrance Ramp from San Felipe Rd. to NB IH 610, Houston

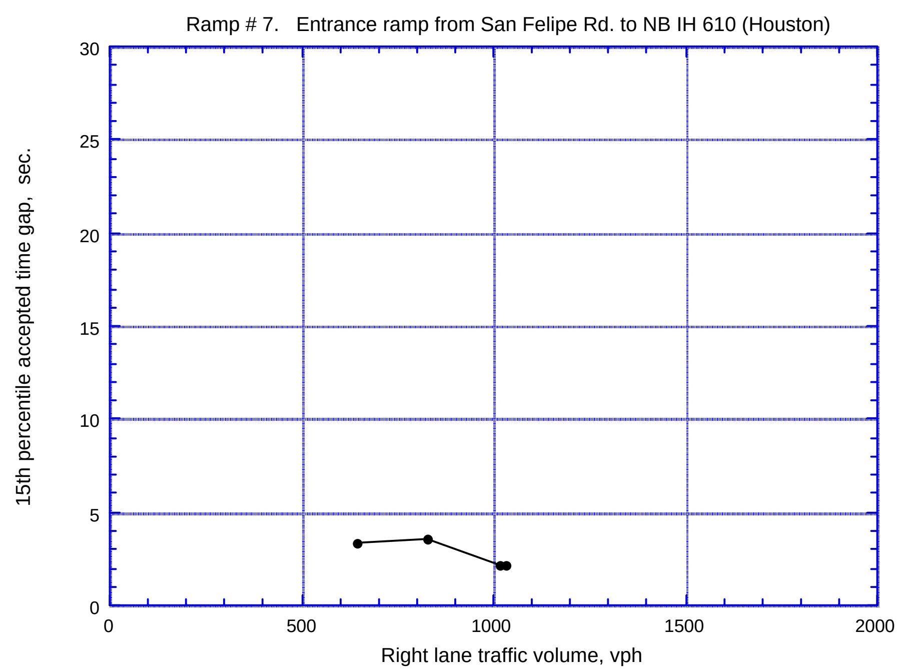

Figure B24 15th Percentile Freeway Time Gap Accepted by Ramp Vehicles, Ramp # 7, Entrance Ramp from San Felipe Rd. to NB IH 610, Houston

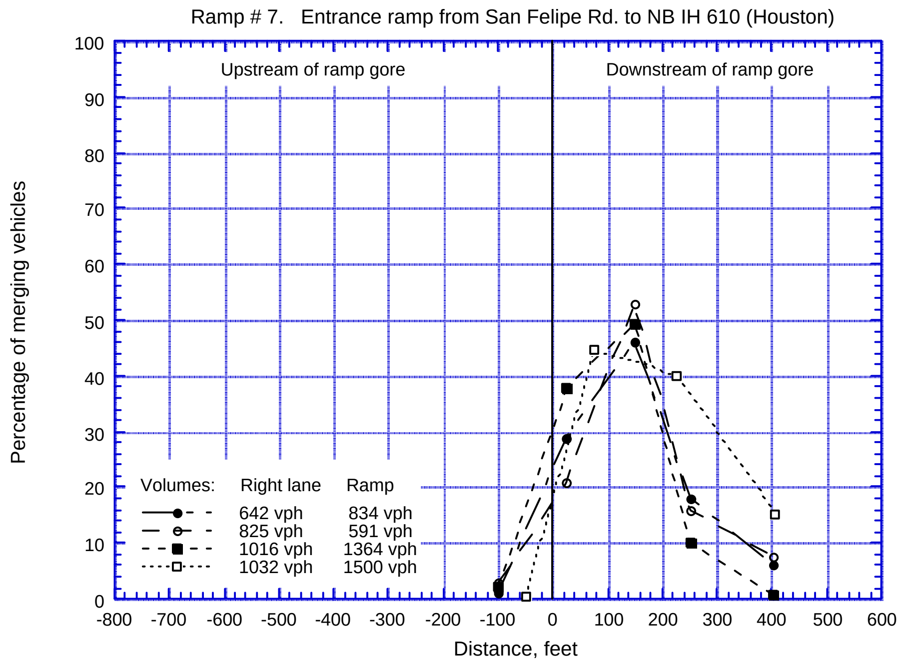

Figure B25 Ramp Vehicle Merging Location Percentage, Ramp # 7,
Entrance Ramp from San Felipe Rd. to NB IH 610, Houston

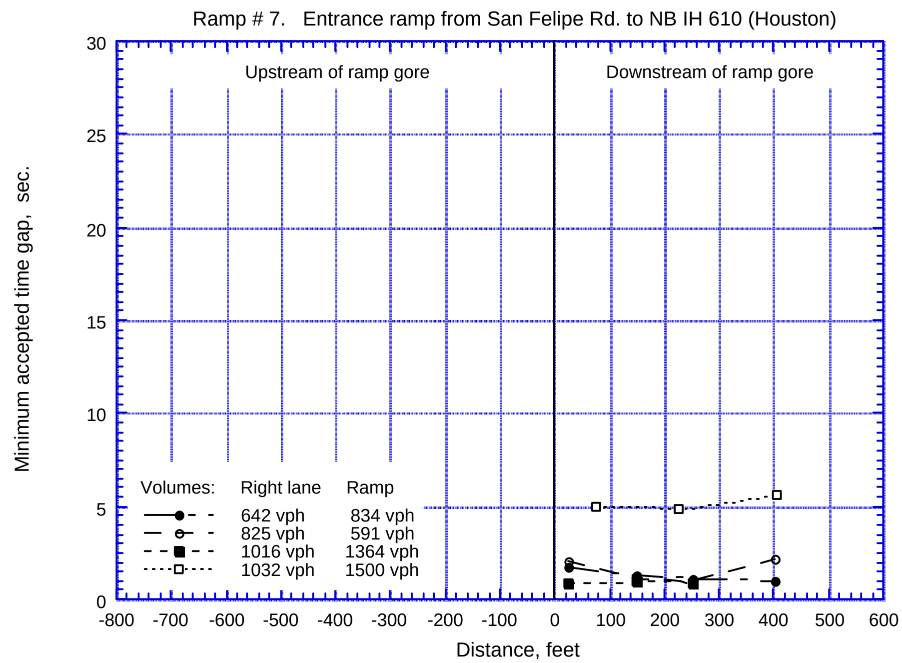

Figure B26 Minimum Time Gap Accepted by Ramp Vehicles, Ramp # 7,
Entrance Ramp from San Felipe Rd. to NB IH 610, Houston

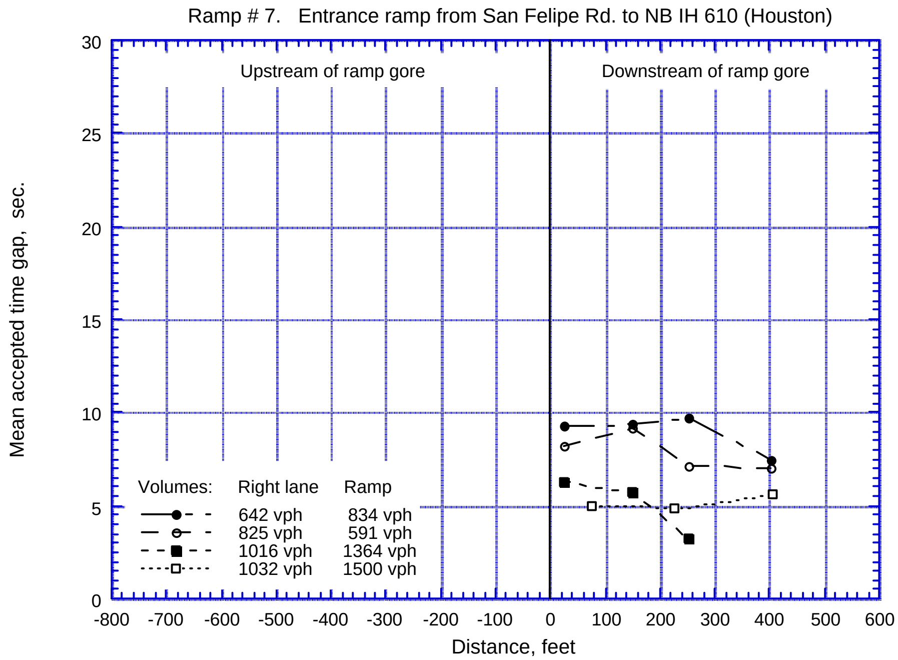

Figure B27 Mean Freeway Time Gap Accepted by Ramp Vehicles Versus Merging Location, Ramp # 7, Entrance Ramp from San Felipe Rd. to NB IH 610, Houston

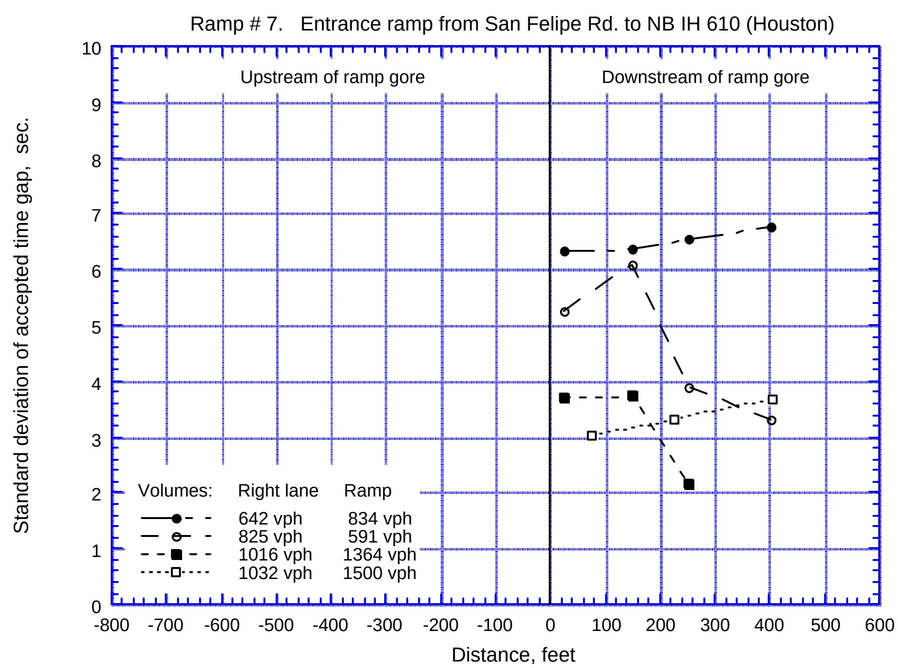

Figure B28 Standard Deviation of Freeway Time Gap Accepted by Ramp Vehicles Versus Merging Location, Ramp # 7, Entrance Ramp from San Felipe Rd. to NB IH 610, Houston

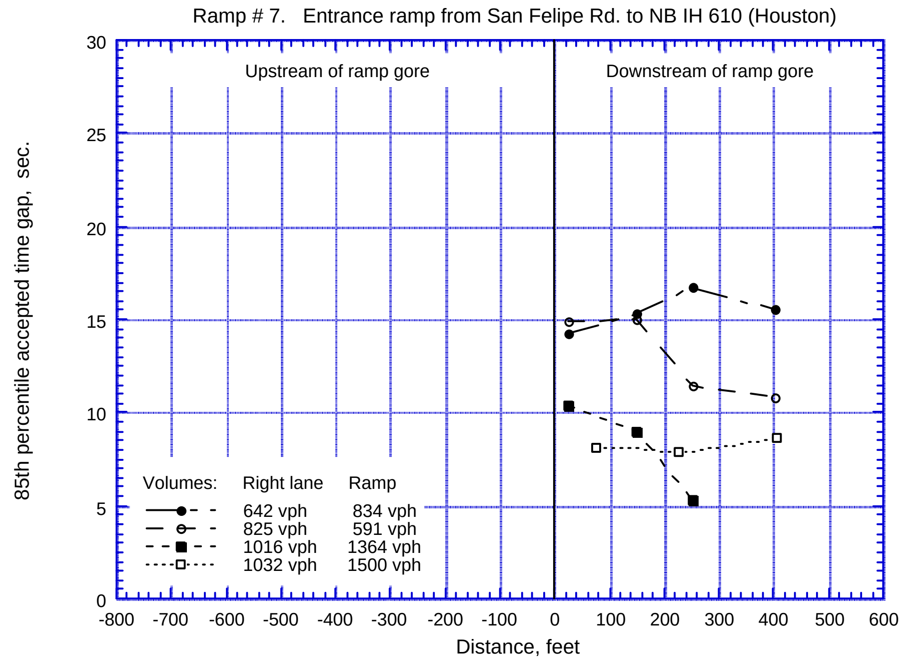

Figure B29 85th Percentile Freeway Time Gap Accepted by Ramp Vehicles Versus Merging Location, Ramp # 7, Entrance Ramp from San Felipe Rd. to NB IH 610, Houston

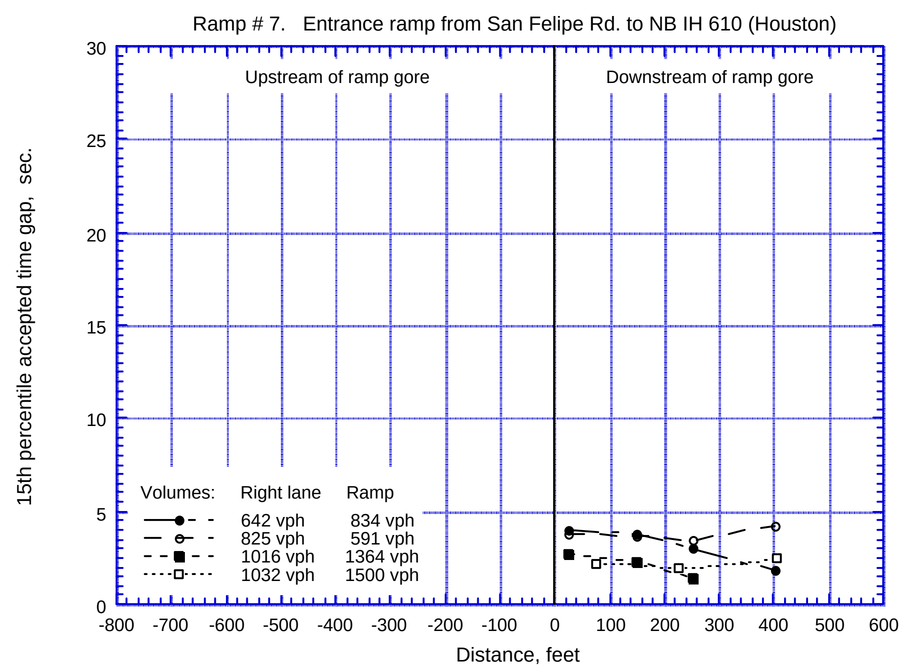

Figure B30 15th Percentile Freeway Time Gap Accepted by Ramp Vehicles Versus Merging Location, Ramp # 7, Entrance Ramp from San Felipe Rd. to NB IH 610, Houston

APPENDIX C

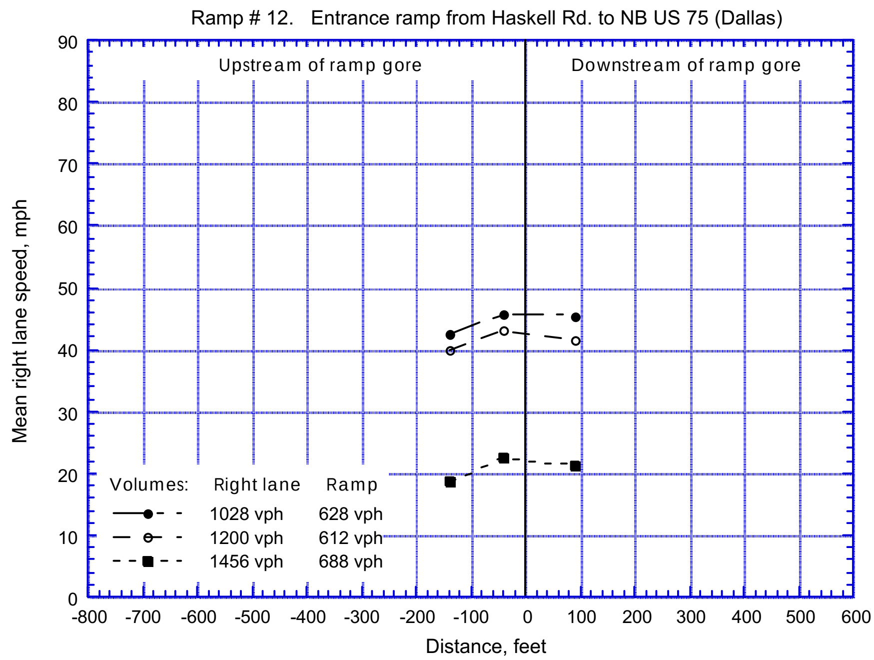

Figure C01 Mean Freeway Right Lane Speed, Ramp #12
Entrance Ramp from Haskell Road to NB US 75, Dallas

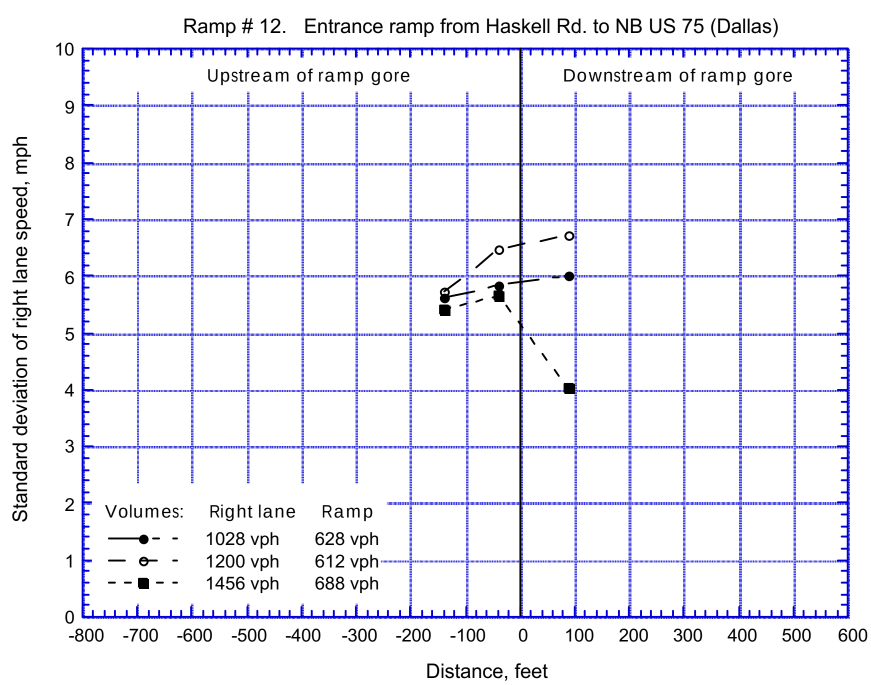

Figure C02 Standard Deviation of Freeway Right Lane Speed, Ramp #12
Entrance Ramp from Haskell Road to NB US 75, Dallas

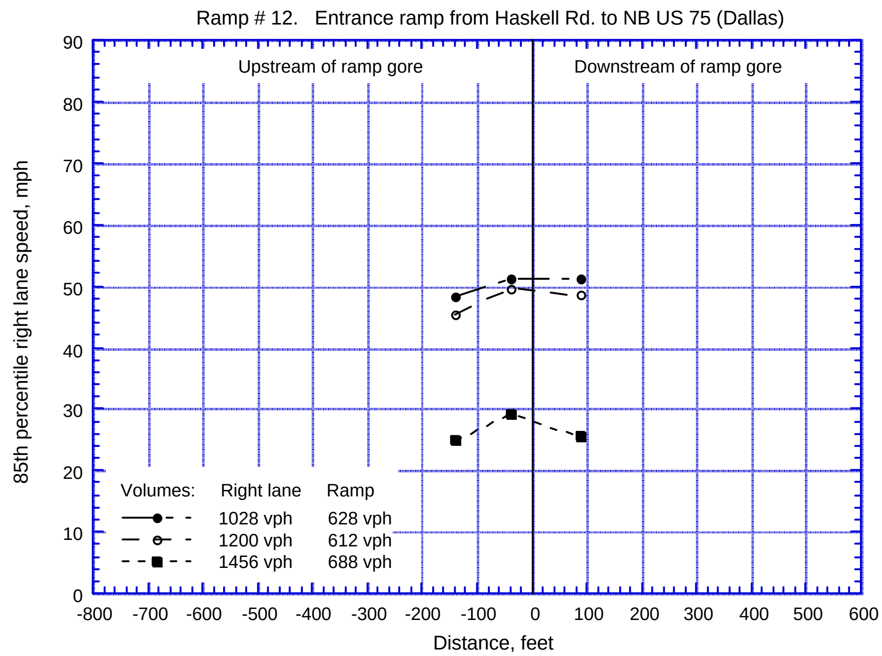

Figure C03 85th Percentile Freeway Right Lane Speed, Ramp #12
Entrance Ramp from Haskell Road to NB US 75, Dallas

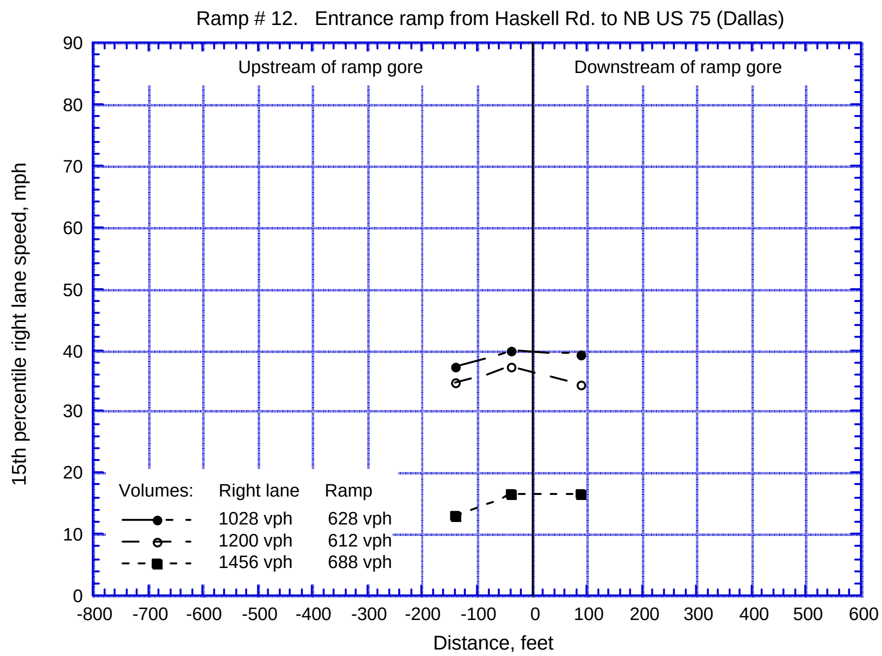

Figure C04 15th Percentile Freeway Right Lane Speed, Ramp #12
Entrance Ramp from Haskell Road to NB US 75, Dallas

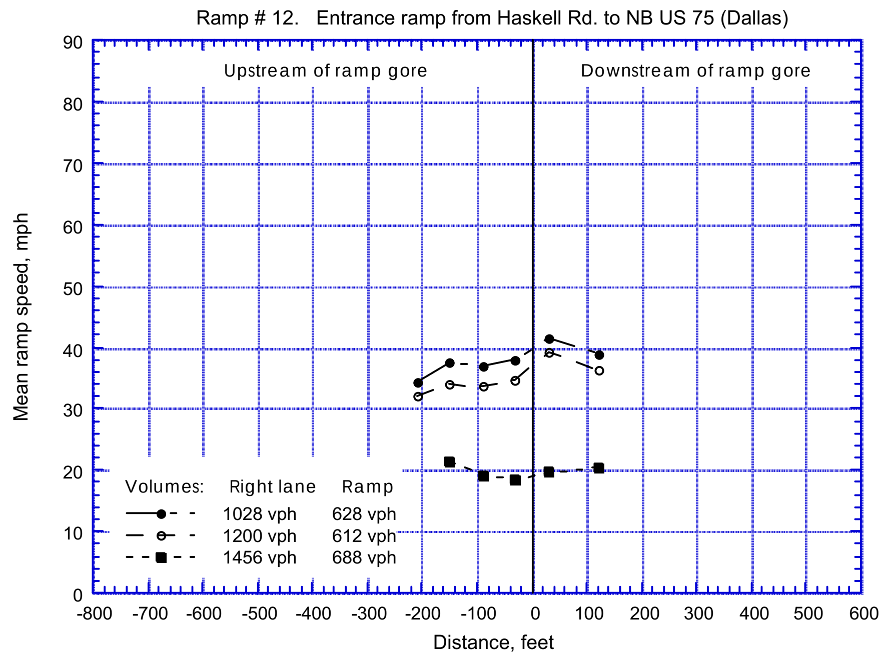

Figure C05 Mean Ramp Speed, Ramp #12 Entrance
Ramp from Haskell Road to NB US 75, Dallas

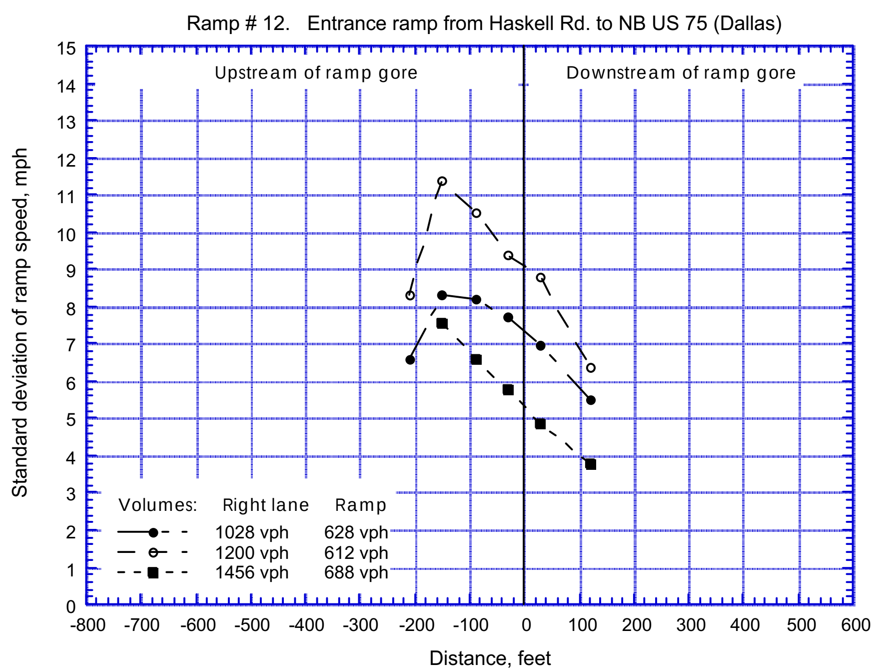

Figure C06 Standard Deviation of Ramp Speed, Ramp #12
Entrance Ramp from Haskell Road to NB US 75, Dallas

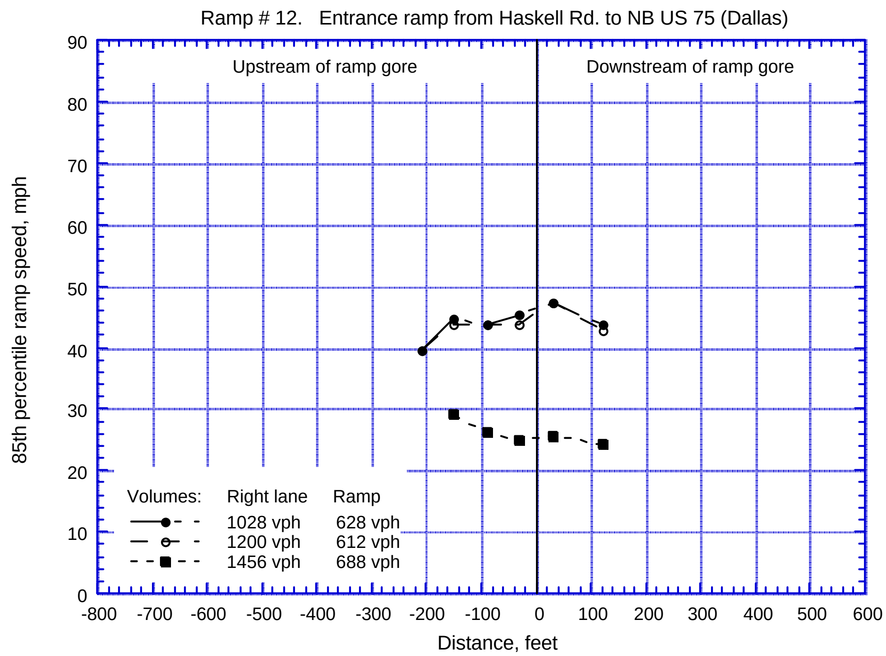

Figure C07 85th Percentile Ramp Speed, Ramp #12 Entrance Ramp from Haskell Road to NB US 75, Dallas

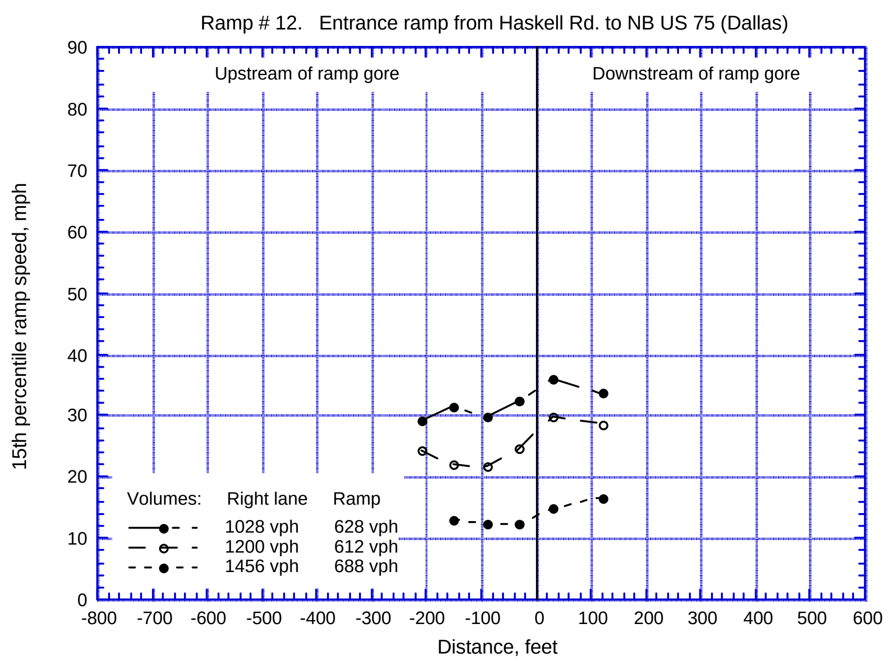

Figure C08 15th Percentile Ramp Speed, Ramp #12 Entrance Ramp from Haskell Road to NB US 75, Dallas

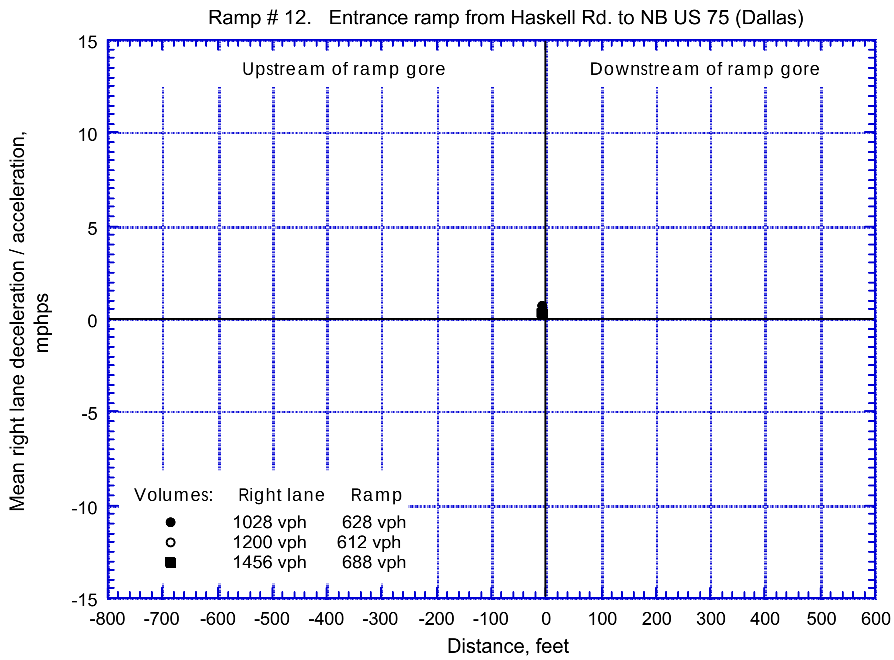

Figure C09 Mean Freeway Right Lane Acceleration/Deceleration, Ramp #12 Entrance Ramp from Haskell Road to NB US 75, Dallas

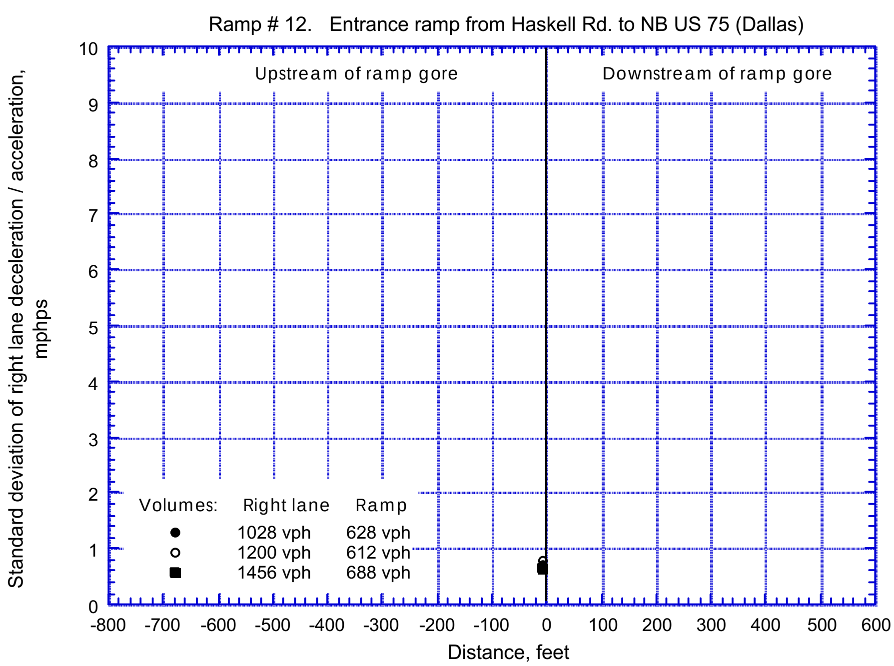

Figure C10 Standard Deviation of Freeway Right Lane Acceleration/Deceleration, Ramp #12 Entrance Ramp from Haskell Road to NB US 75, Dallas

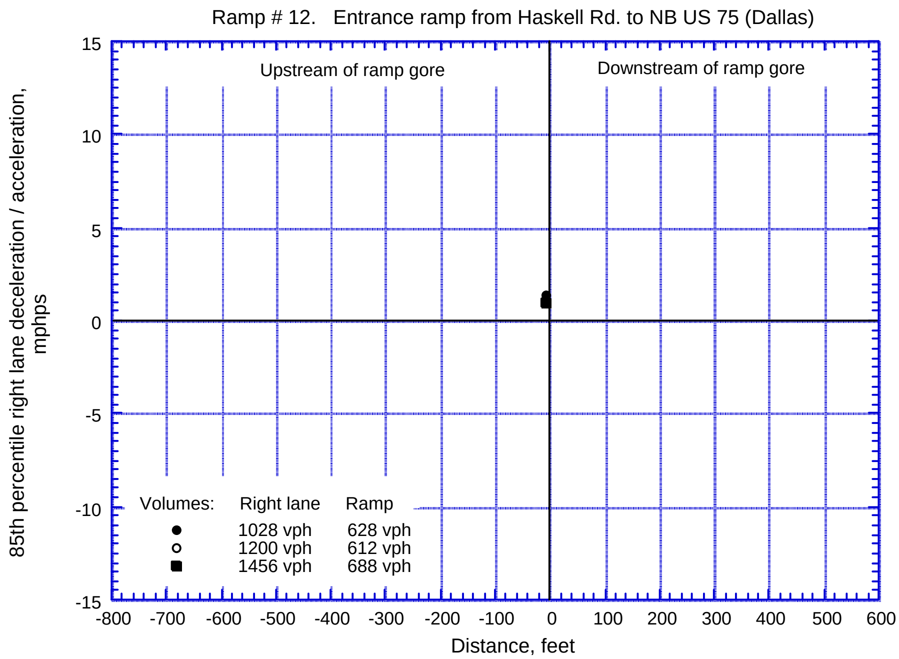

Figure C11 85th Percentile Freeway Right Lane Acceleration/Deceleration, Ramp #12 Entrance Ramp from Haskell Road to NB US 75, Dallas

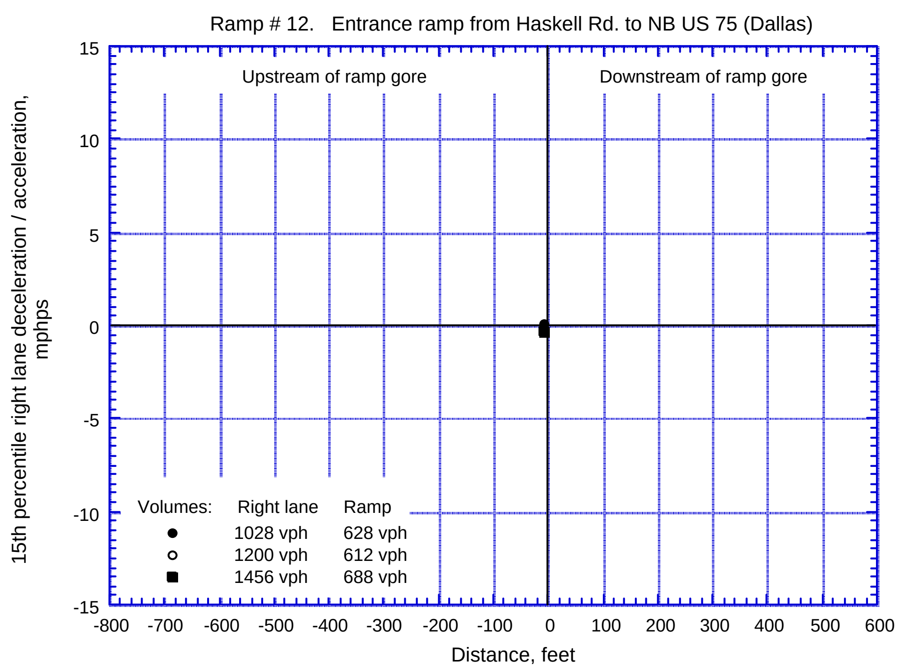

Figure C12 15th Percentile Freeway Right Lane Acceleration/Deceleration, Ramp #12 Entrance Ramp from Haskell Road to NB US 75, Dallas

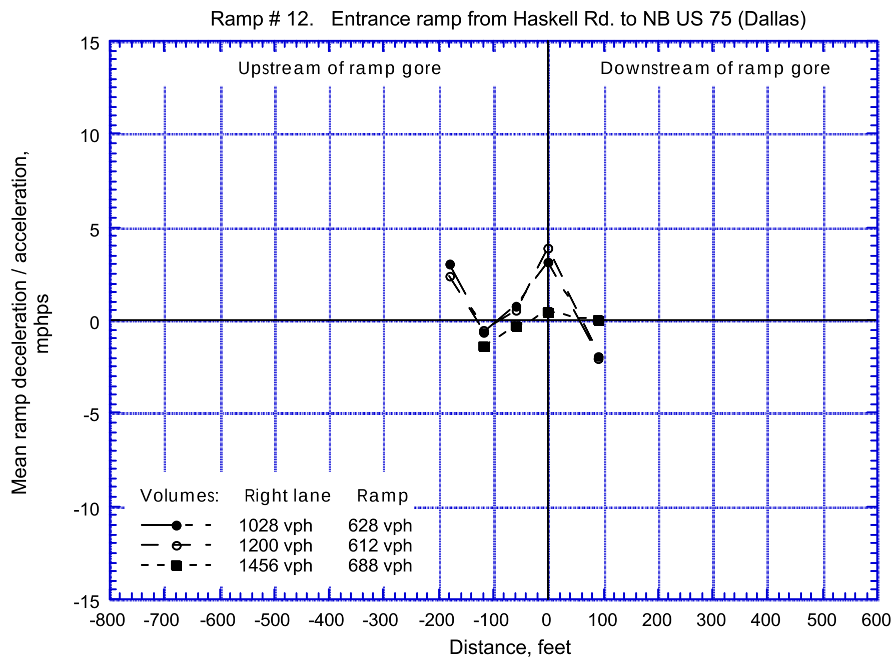

Figure C13 Mean Ramp Acceleration/Deceleration, Ramp #12
Entrance Ramp from Haskell Road to NB US 75, Dallas

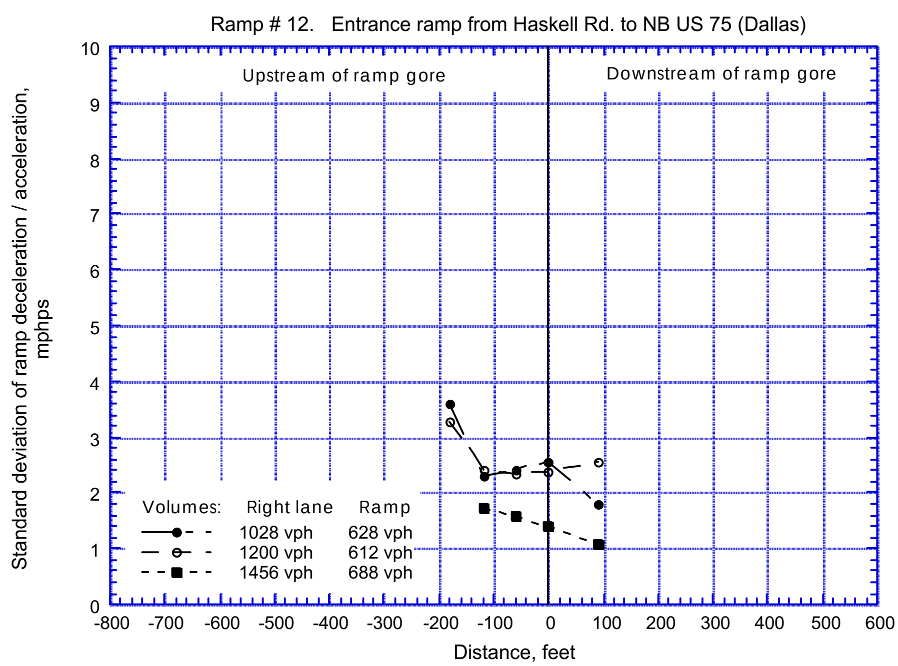

Figure C14 Standard Deviation of Ramp Acceleration/Deceleration,
Ramp #12 Entrance Ramp from Haskell Road to NB US 75, Dallas

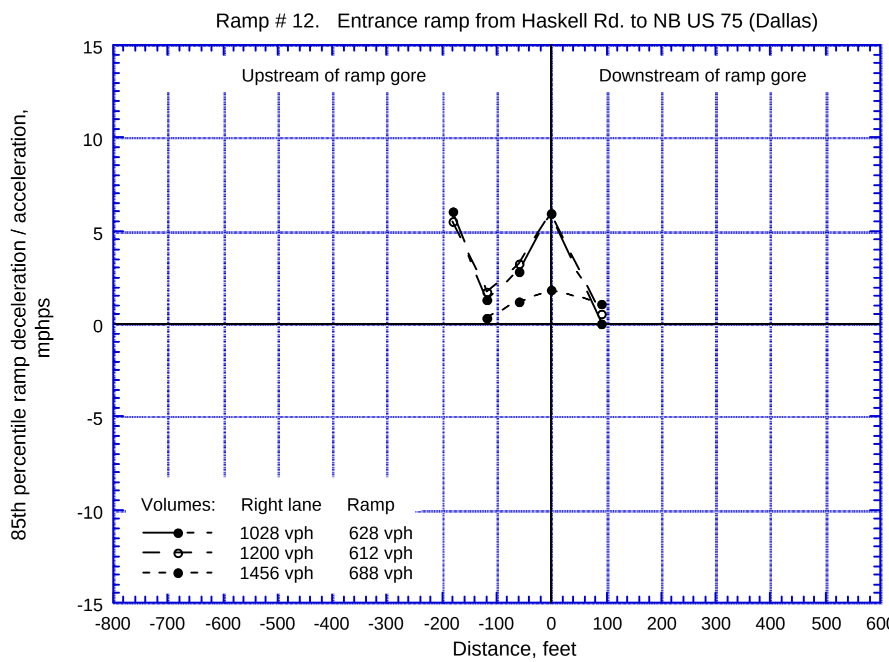

Figure C15 85th Percentile Ramp Acceleration/Deceleration, Ramp #12
Entrance Ramp from Haskell Road to NB US 75, Dallas

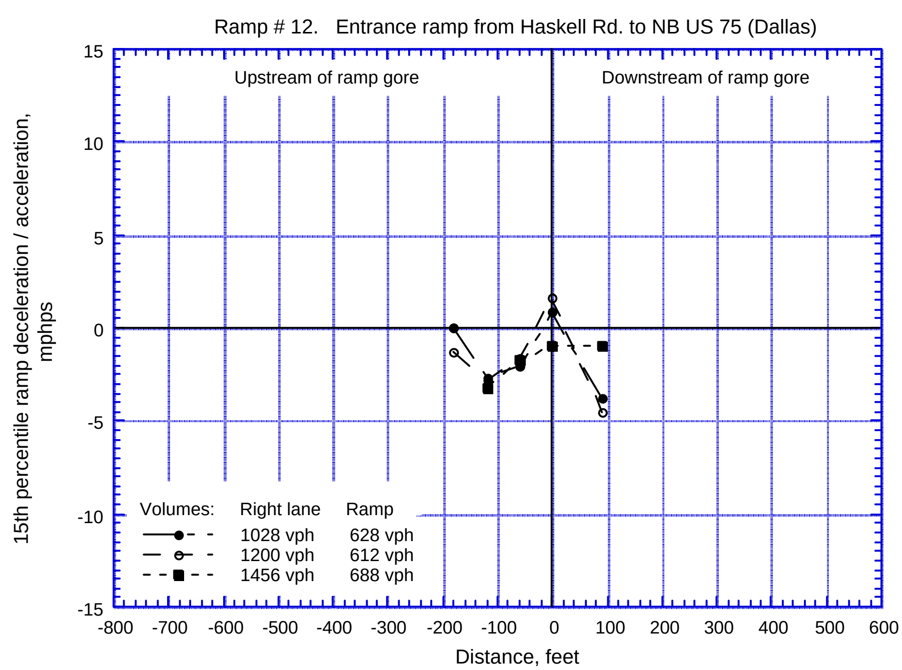

Figure C16 15th Percentile Ramp Acceleration/Deceleration, Ramp #12
Entrance Ramp from Haskell Road to NB US 75, Dallas

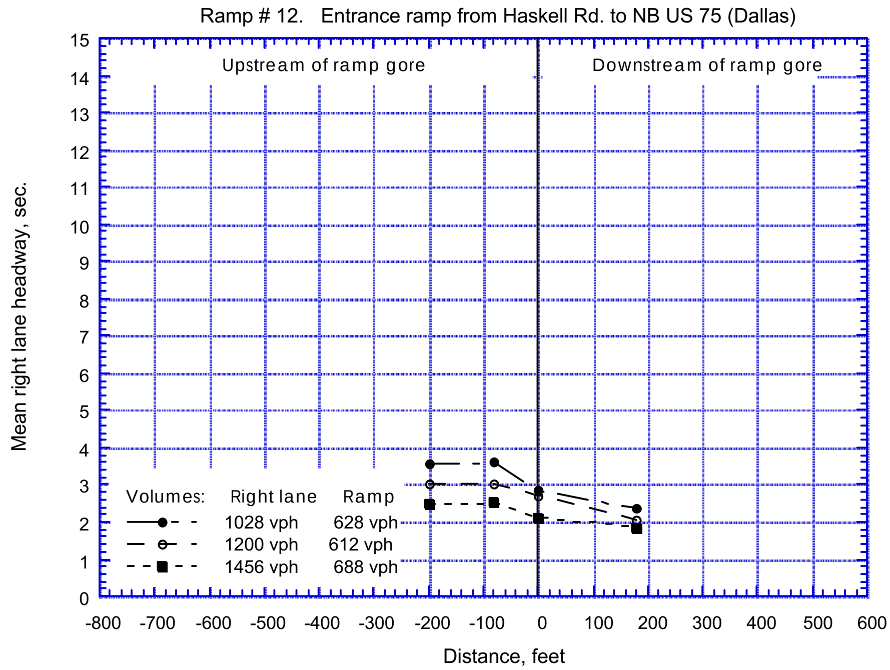

Figure C17 Mean Time Headway Freeway Right Lane, Ramp #12
Entrance Ramp from Haskell Road to NB US 75, Dallas

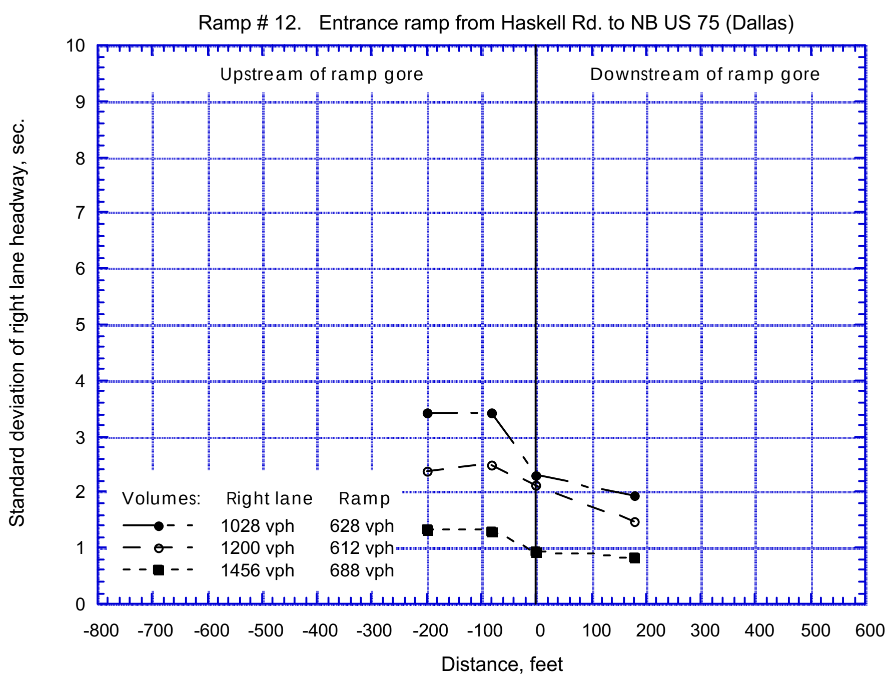

Figure C18 Standard Deviation of Time Headway Freeway Right Lane,
Ramp #12 Entrance Ramp from Haskell Road to NB US 75, Dallas

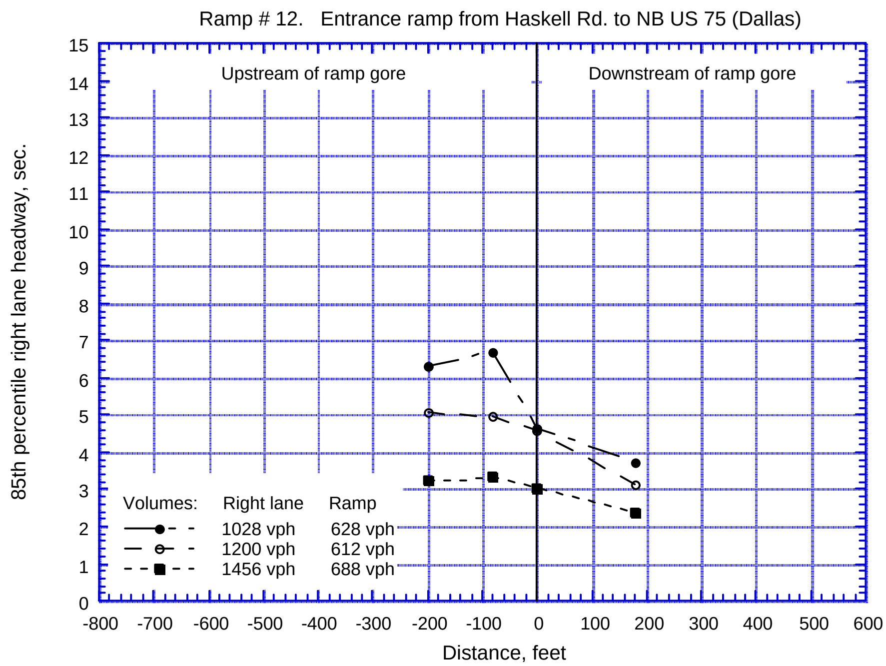

Figure C19 85th Percentile Time Headway Freeway Right Lane, Ramp #12
Entrance Ramp from Haskell Road to NB US 75, Dallas

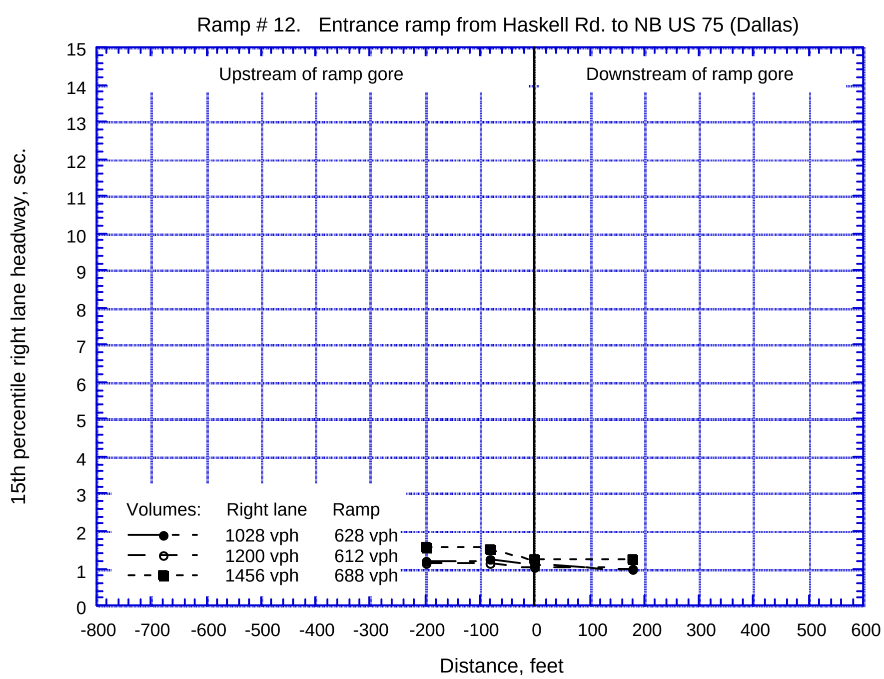

Figure C20 15th Percentile Time Headway Freeway Right Lane, Ramp #12
Entrance Ramp from Haskell Road to NB US 75, Dallas

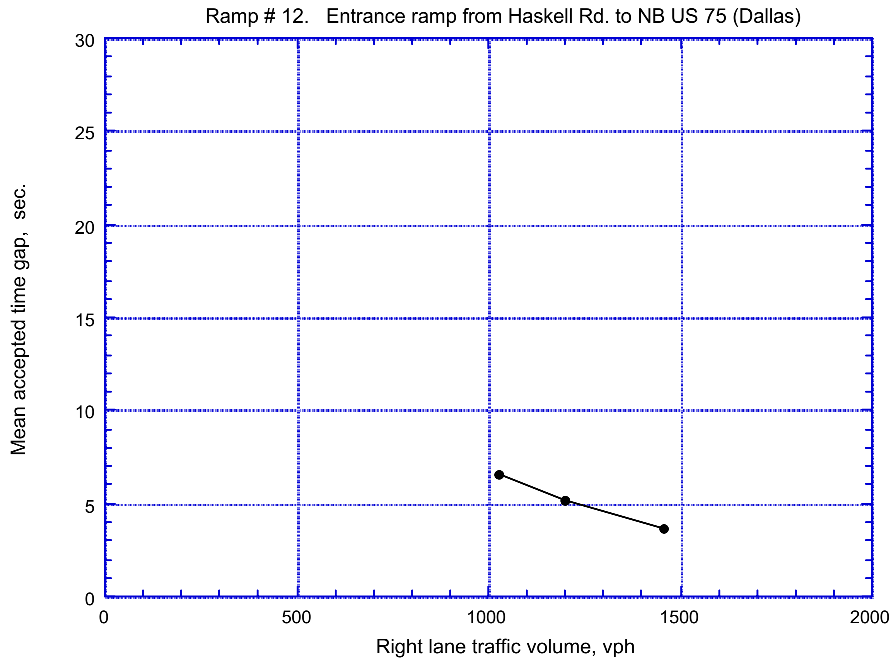

Figure C21 Mean Freeway Time Gap Accepted by Ramp Vehicles Versus Right Freeway Lane Traffic Volume, Ramp #12 Entrance Ramp from Haskell Road to NB US 75, Dallas

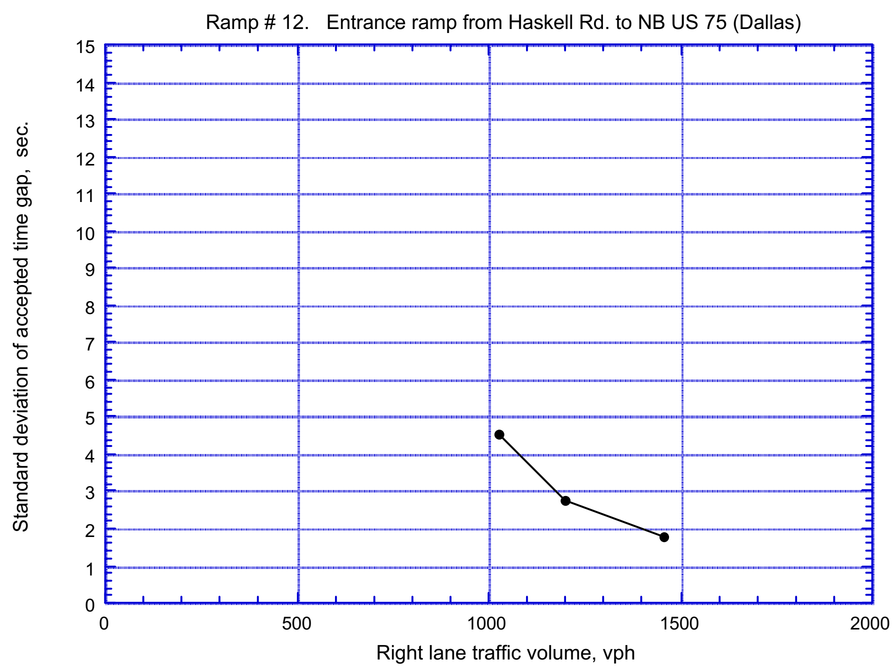

Figure C22 Standard Deviation of Freeway Time Gap Accepted by Ramp Vehicles, Ramp #12 Entrance Ramp from Haskell Road to NB US 75, Dallas

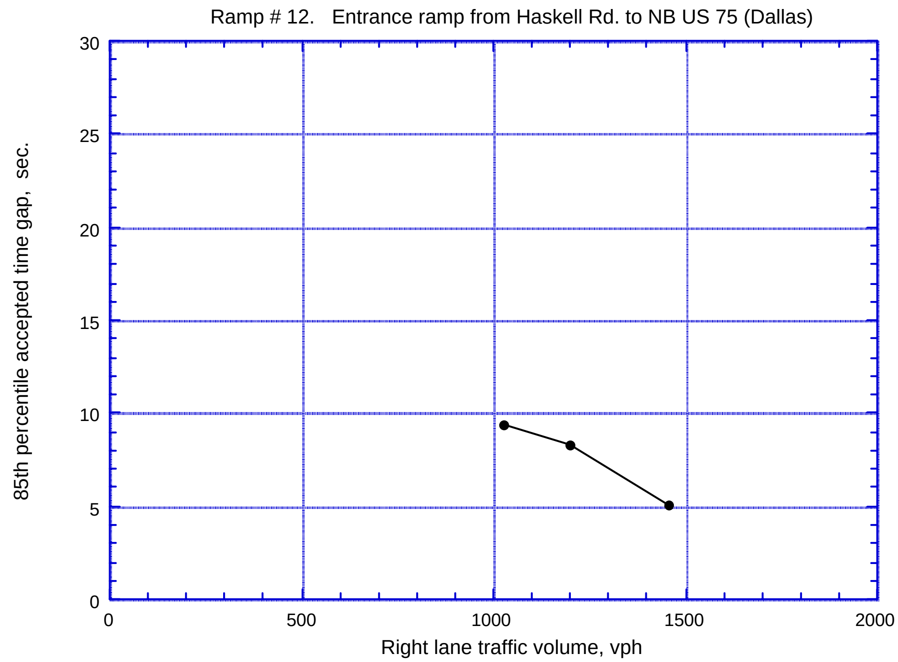

Figure C23 85th Percentile Freeway Time Gap Accepted by Ramp Vehicles, Ramp #12 Entrance Ramp from Haskell Road to NB US 75, Dallas

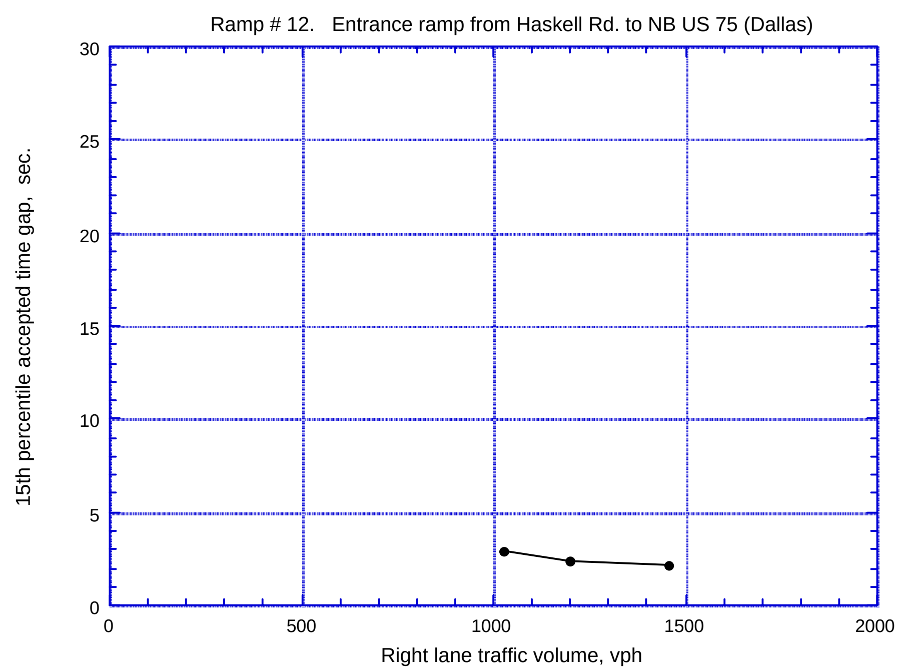

Figure C24 15th Percentile Freeway Time Gap Accepted by Ramp Vehicles, Ramp #12 Entrance Ramp from Haskell Road to NB US 75, Dallas

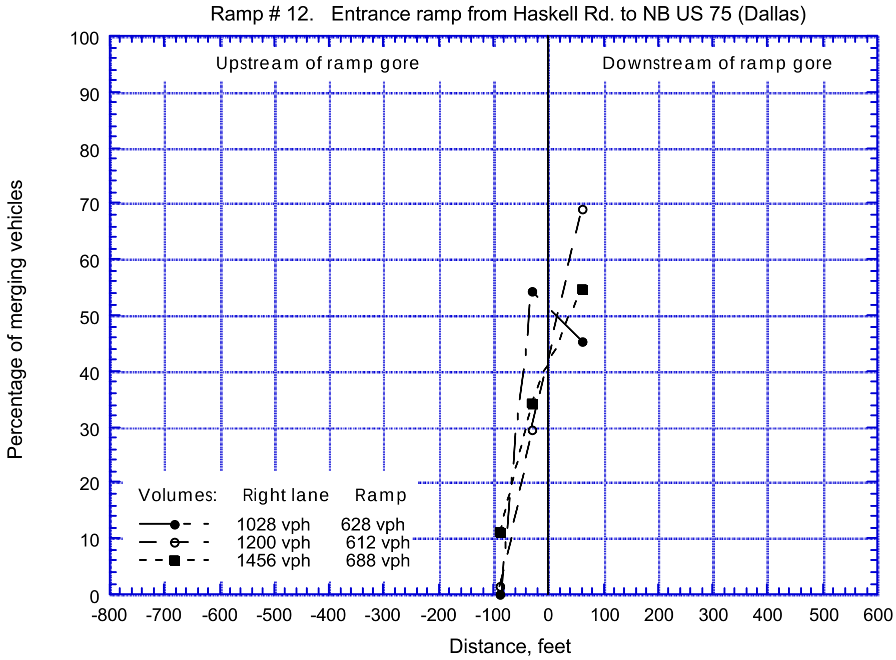

Figure C25 Ramp Vehicle Merging Location Percentage, Ramp #12
Entrance Ramp from Haskell Road to NB US 75, Dallas

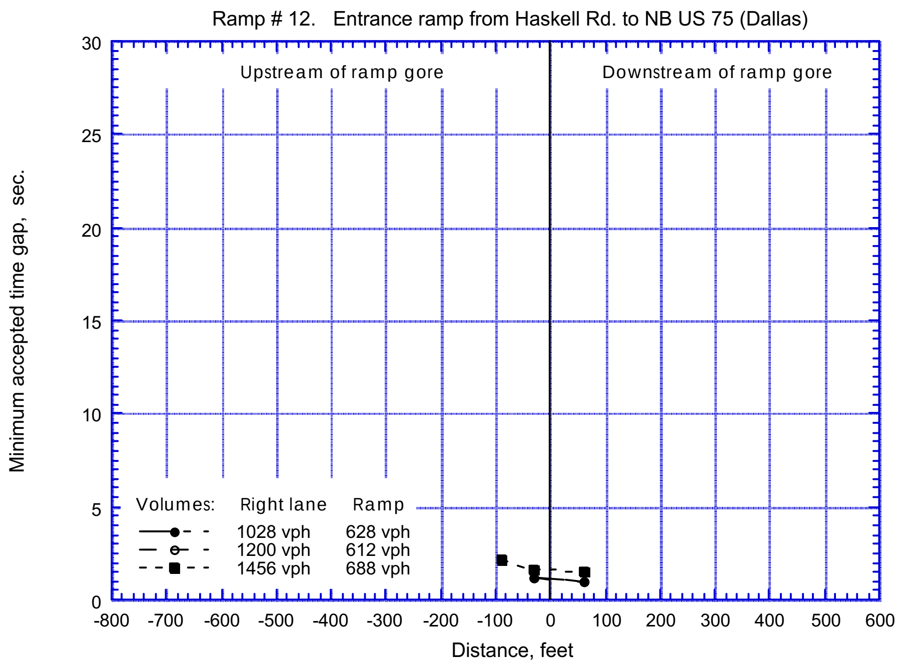

Figure C26 Minimum Time Gap Accepted by Ramp Vehicles, Ramp #12
Entrance Ramp from Haskell Road to NB US 75, Dallas

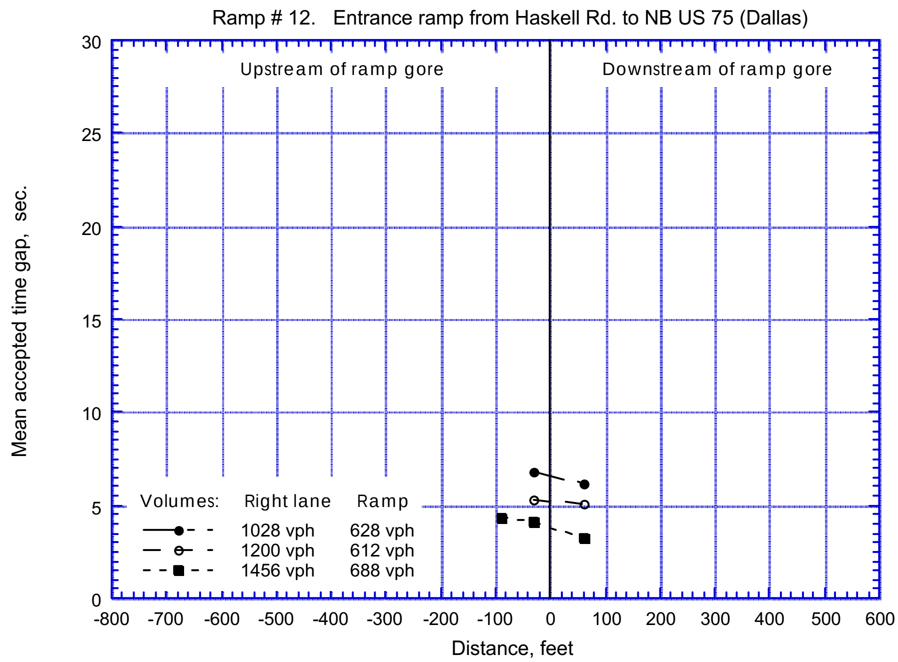

Figure C27 Mean Freeway Time Gap Accepted by Ramp Vehicles Versus Merging Location, Ramp #12 Entrance Ramp from Haskell Road to NB US 75, Dallas

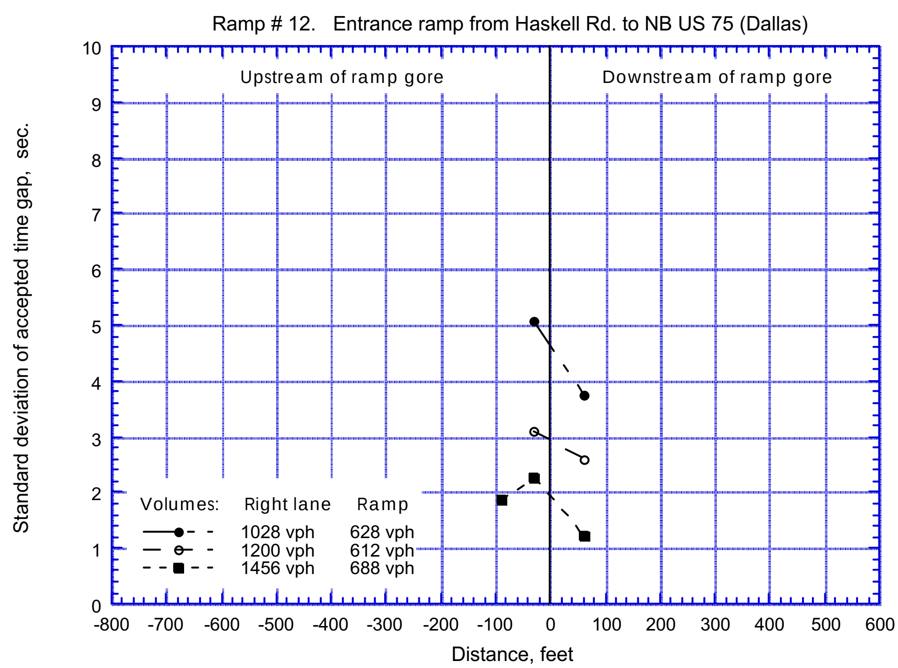

Figure C28 Standard Deviation of Freeway Time Gap Accepted by Ramp Vehicles Versus Merging Location, Ramp #12 Entrance Ramp from Haskell Road to NB US 75, Dallas

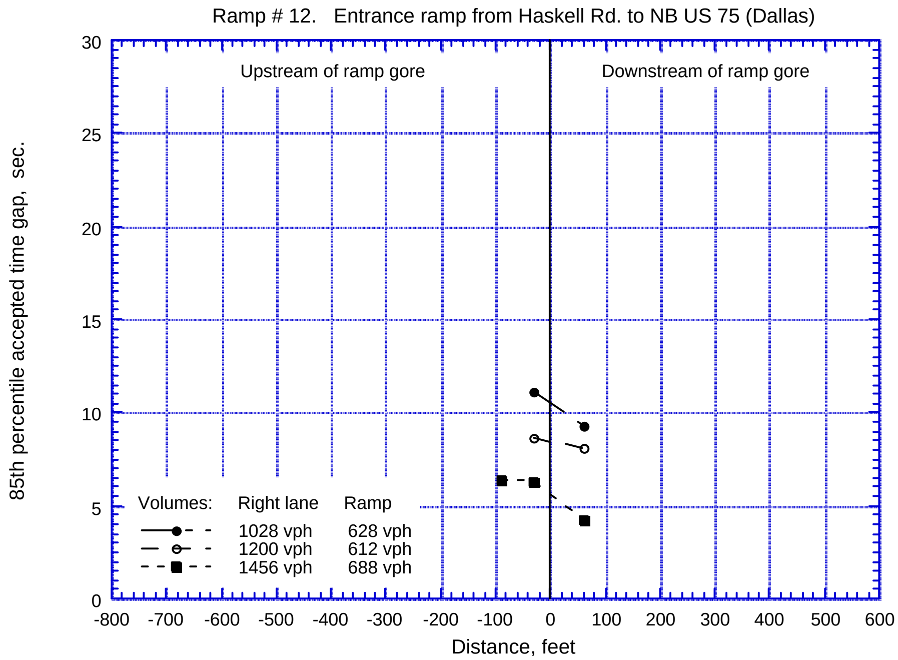

Figure C29 85th Percentile Freeway Time Gap Accepted by Ramp Vehicles Versus Merging Location, Ramp #12 Entrance Ramp from Haskell Road to NB US 75, Dallas

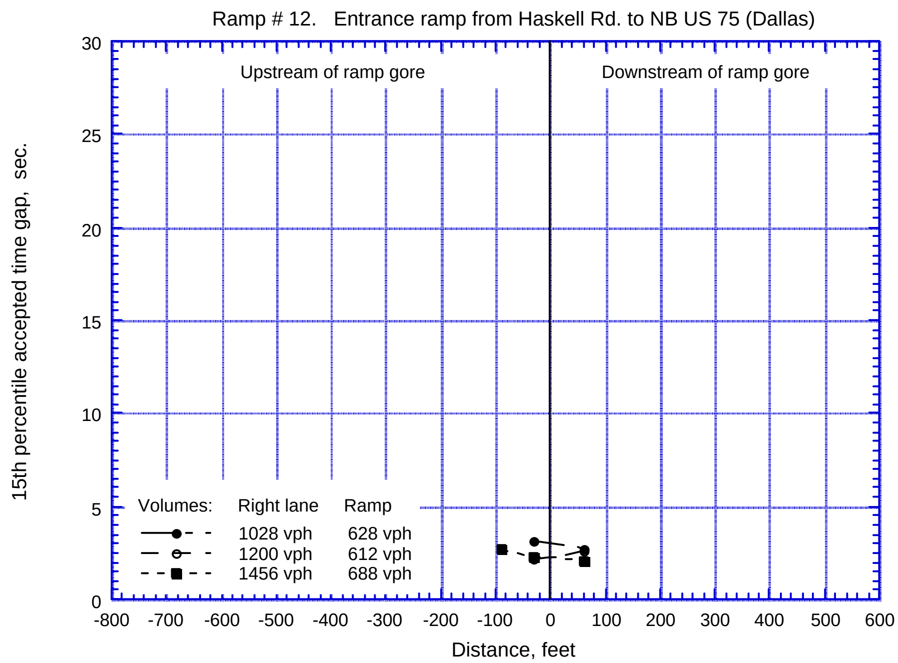

Figure C30 15th Percentile Freeway Time Gap Accepted by Ramp Vehicles Versus Merging Location, Ramp #12 Entrance Ramp from Haskell Road to NB US 75, Dallas

APPENDIX D

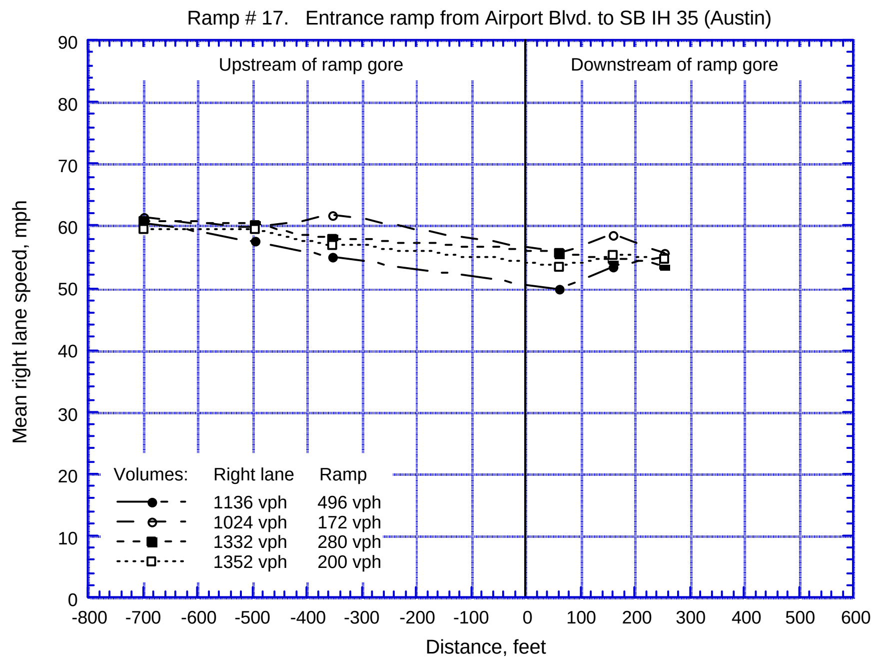

Figure D01 Mean Freeway Right Lane Speed, Ramp #17
Entrance Ramp from Airport Blvd. to SB IH 35, Austin

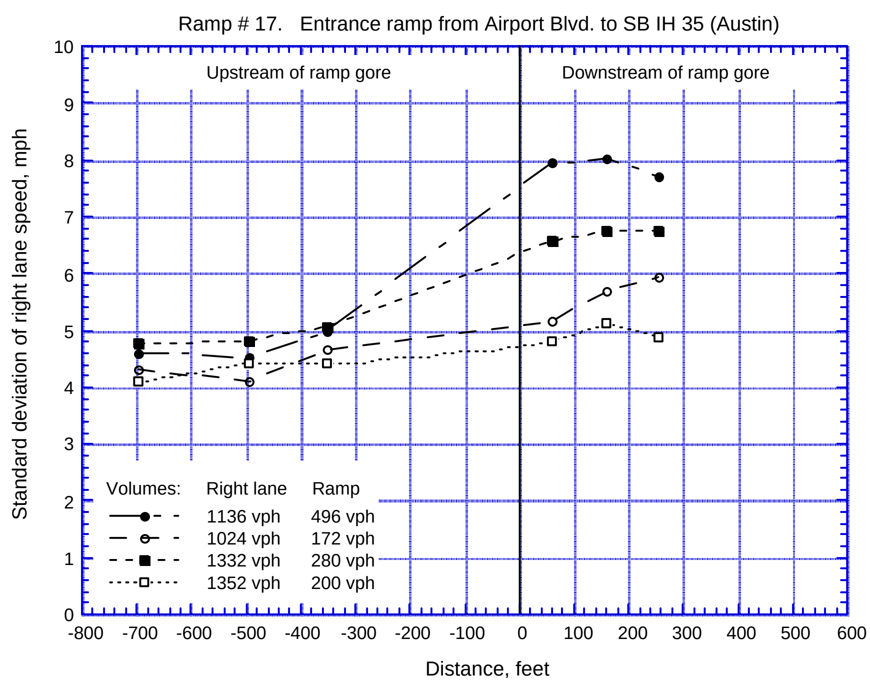

Figure D02 Standard Deviation of Freeway Right Lane Speed,
Ramp #17 Entrance Ramp from Airport Blvd. to SB IH 35, Austin

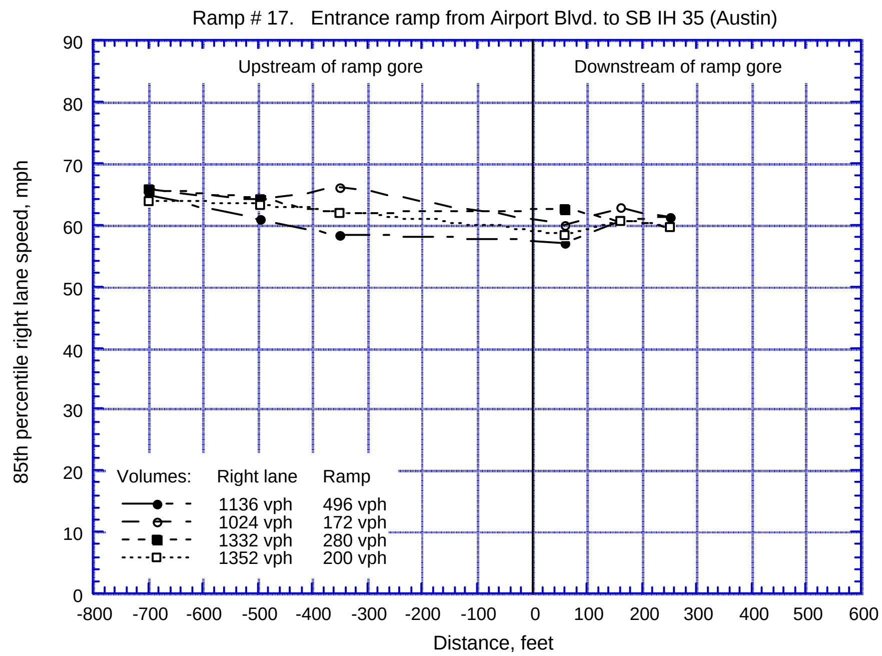

Figure D03 85th Percentile Freeway Right Lane Speed, Ramp #17
Entrance Ramp from Airport Blvd. to SB IH 35, Austin

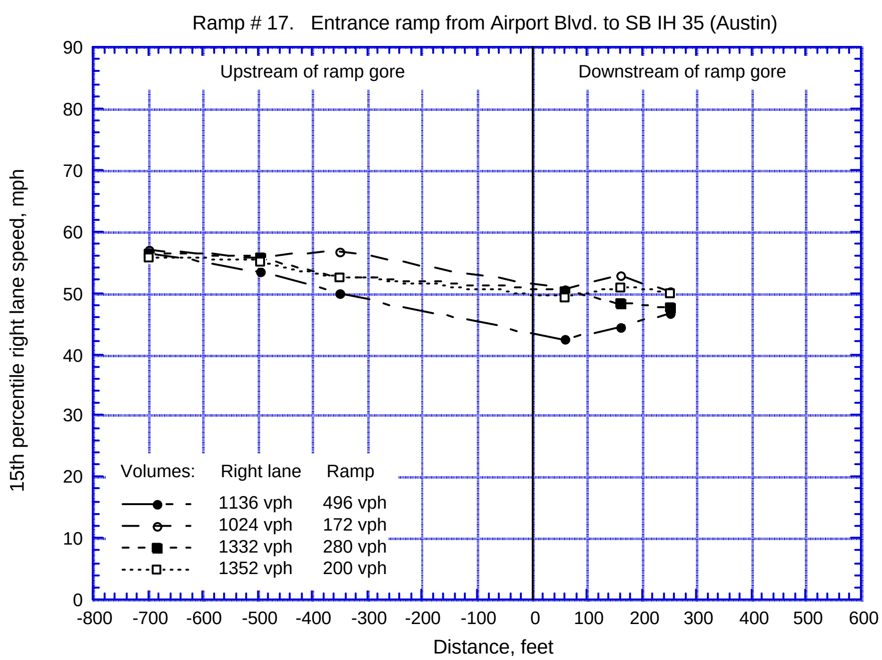

Figure D04 15th Percentile Freeway Right Lane Speed, Ramp #17
Entrance Ramp from Airport Blvd. to SB IH 35, Austin

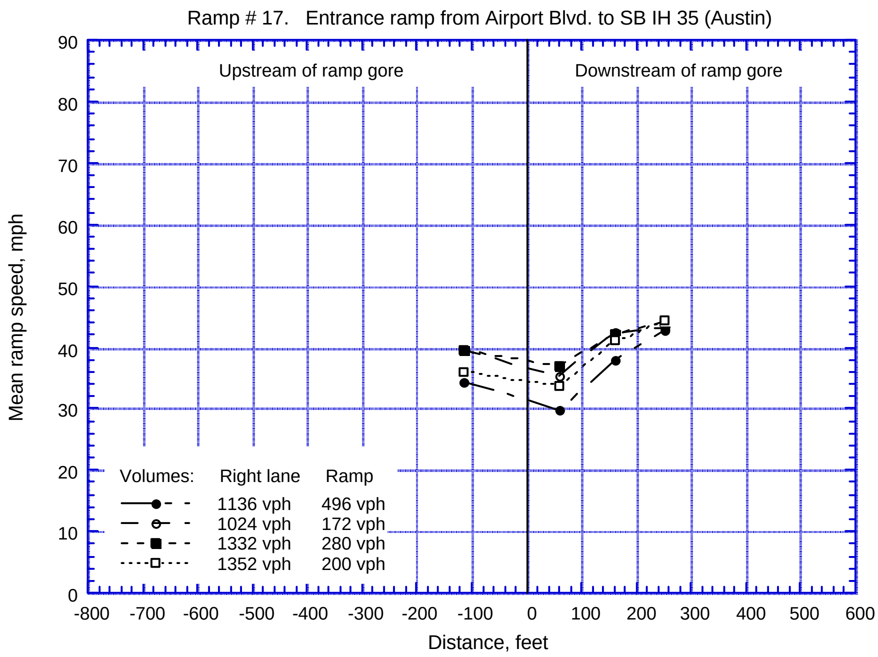

Figure D05 Mean Ramp Speed, Ramp #17 Entrance
Ramp from Airport Blvd. to SB IH 35, Austin

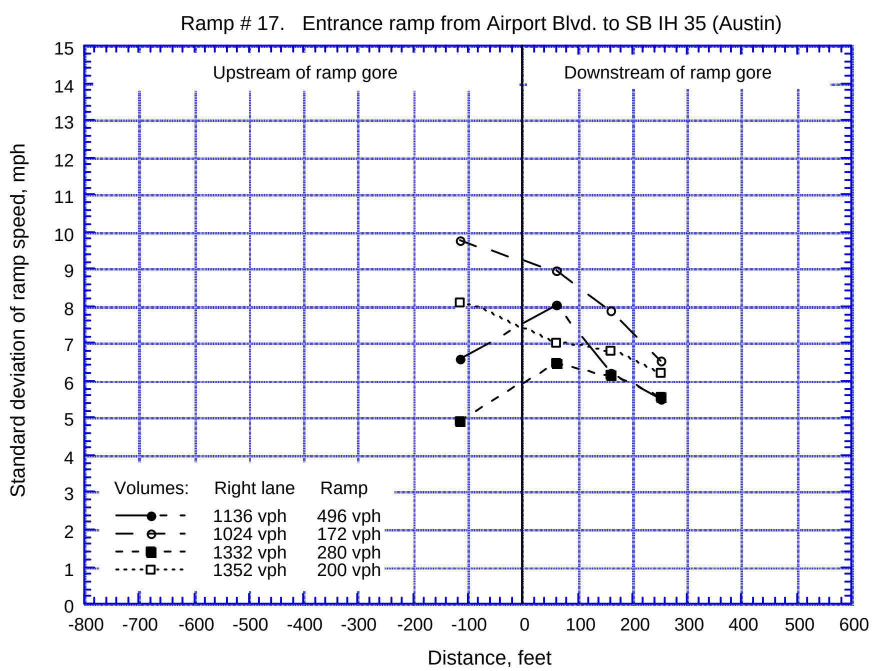

Figure D06 Standard Deviation of Ramp Speed, Ramp #17
Entrance Ramp from Airport Blvd. to SB IH 35, Austin

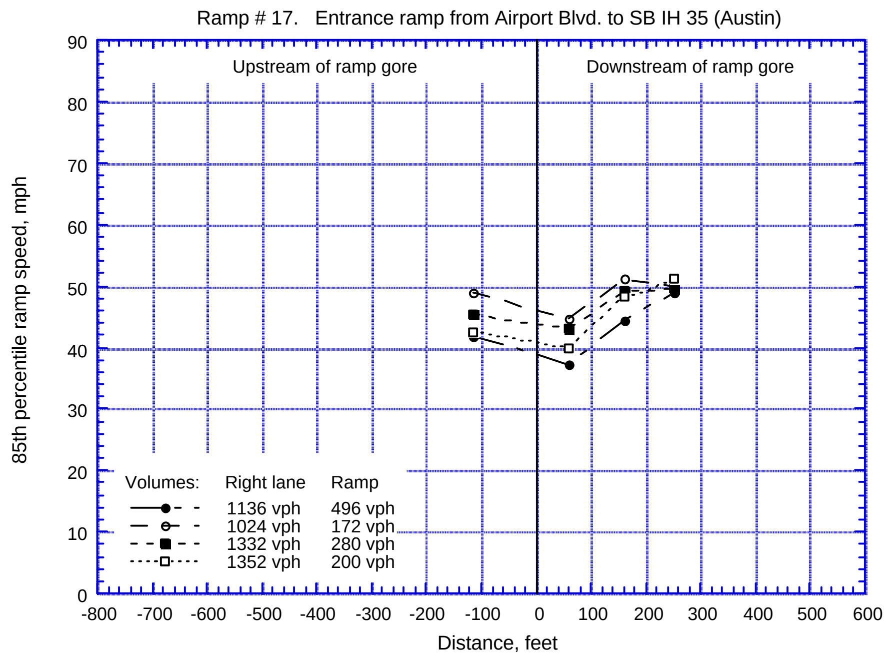

Figure D07 85th Percentile Ramp Speed, Ramp #17
Entrance Ramp from Airport Blvd. to SB IH 35, Austin

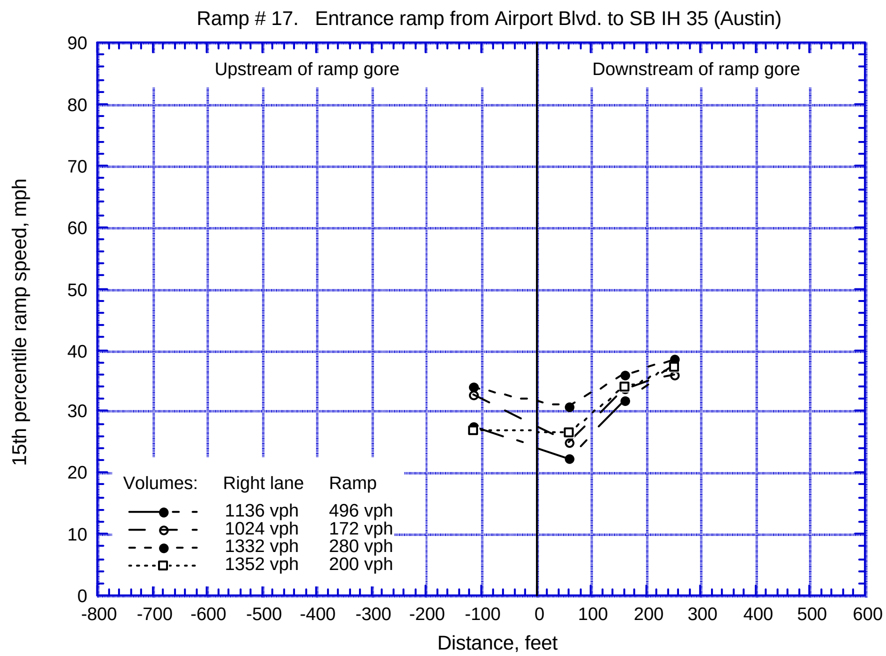

Figure D08 15th Percentile Ramp Speed, Ramp #17
Entrance Ramp from Airport Blvd. to SB IH 35, Austin

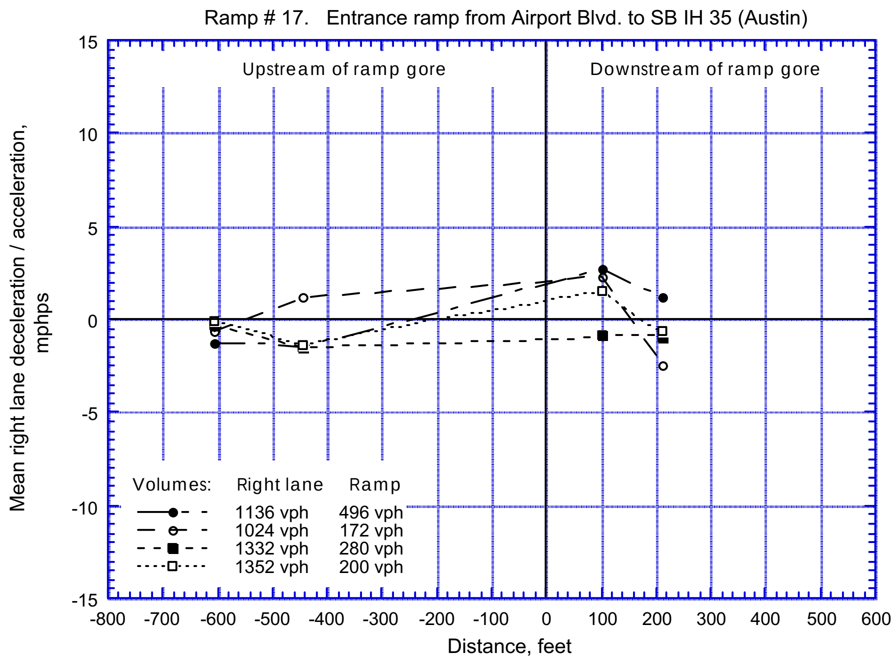

Figure D09 Mean Freeway Right Lane Acceleration/Deceleration,
Ramp #17 Entrance Ramp from Airport Blvd. to SB IH 35, Austin

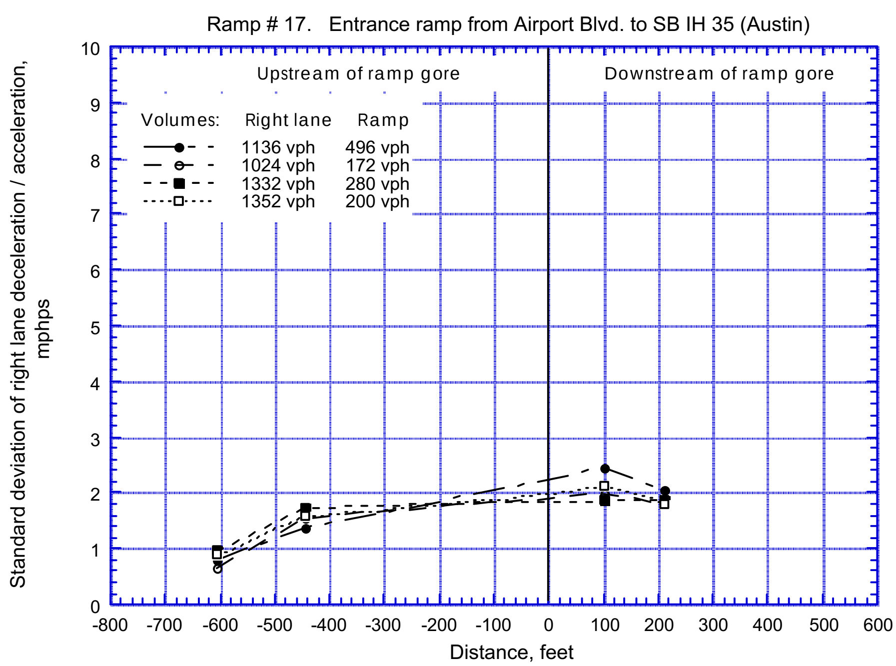

Figure D10 Standard Deviation of Freeway Right Lane Acceleration/Deceleration,
Ramp #17 Entrance Ramp from Airport Blvd. to SB IH 35, Austin

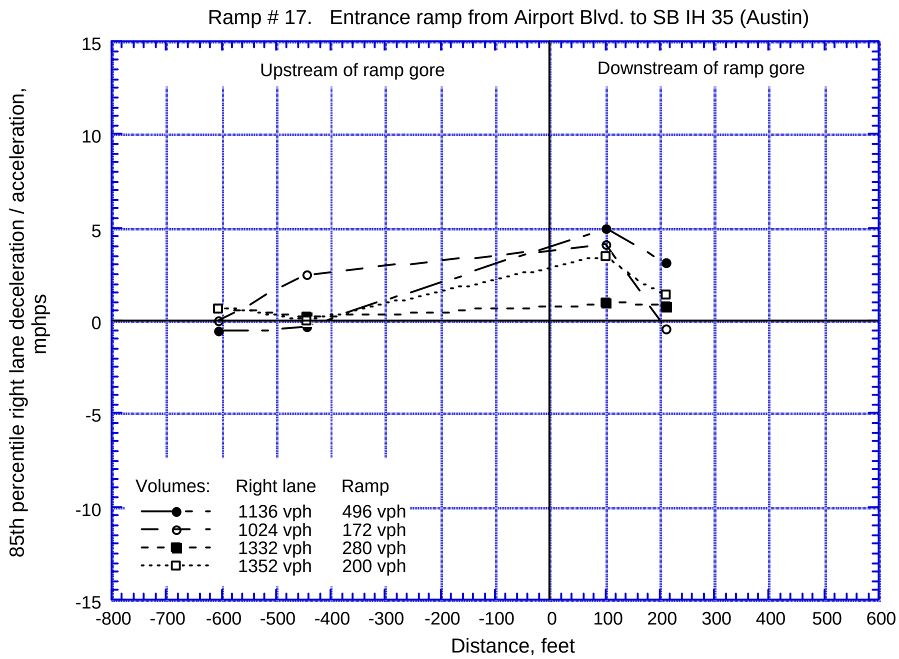

Figure D11 85th Percentile Freeway Right Lane Acceleration/Deceleration, Ramp #17 Entrance Ramp from Airport Blvd. to SB IH 35, Austin

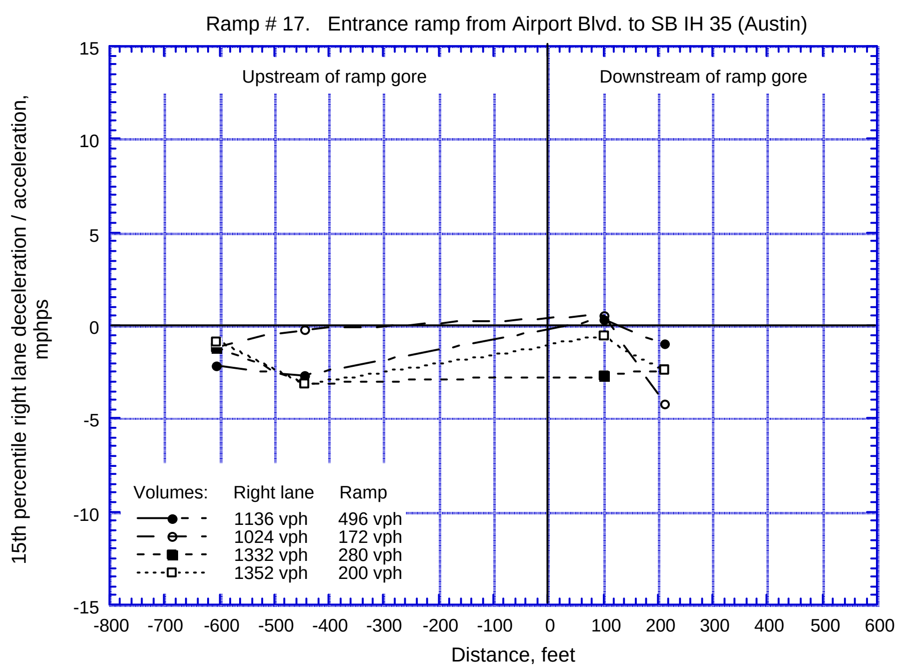

Figure D12 15th Percentile Freeway Right Lane Acceleration/Deceleration, Ramp #17 Entrance Ramp from Airport Blvd. to SB IH 35, Austin

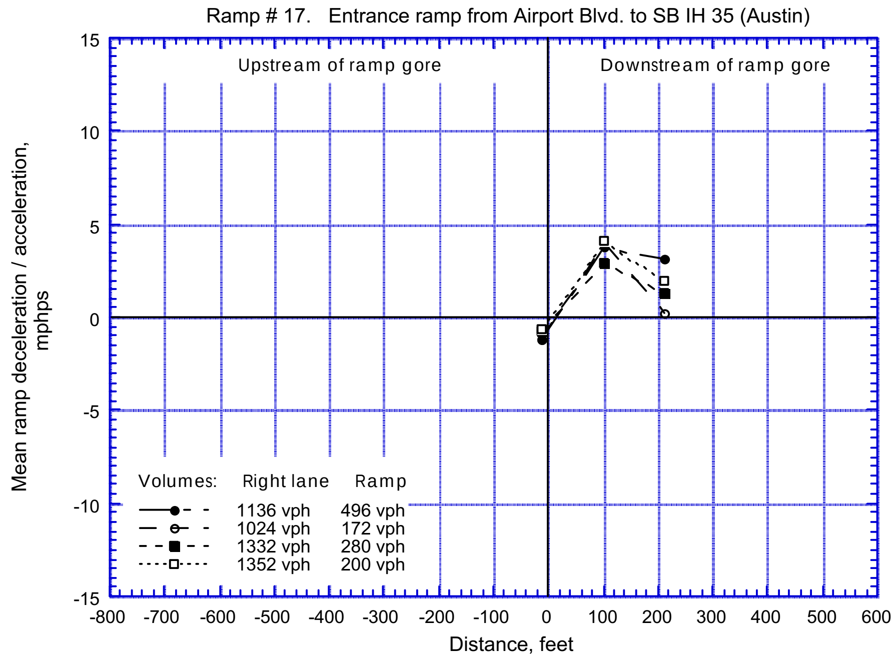

Figure D13 Mean Ramp Acceleration/Deceleration, Ramp #17
Entrance Ramp from Airport Blvd. to SB IH 35, Austin

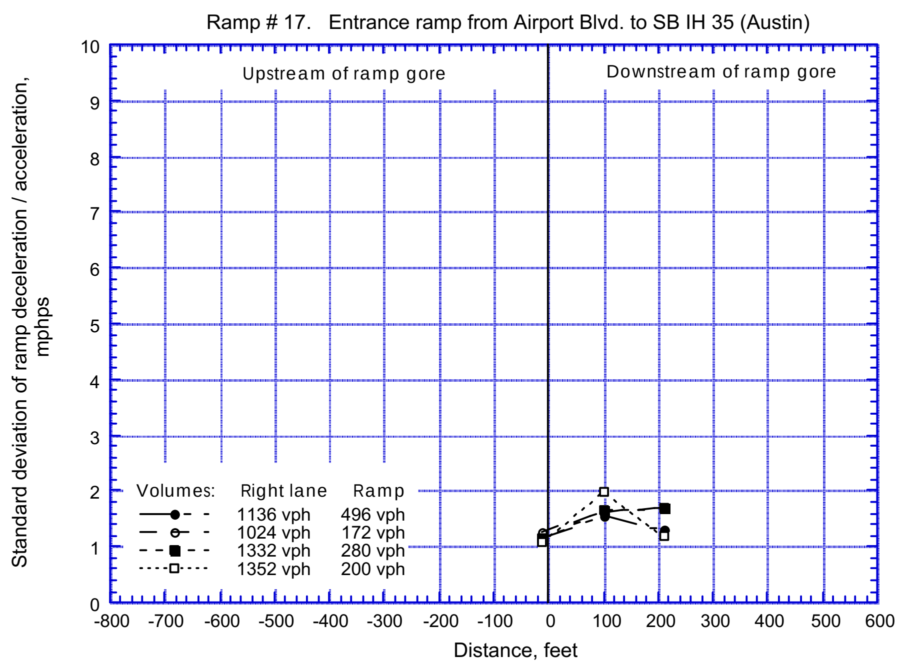

Figure D14 Standard Deviation of Ramp Acceleration/Deceleration,
Ramp #17 Entrance Ramp from Airport Blvd. to SB IH 35, Austin

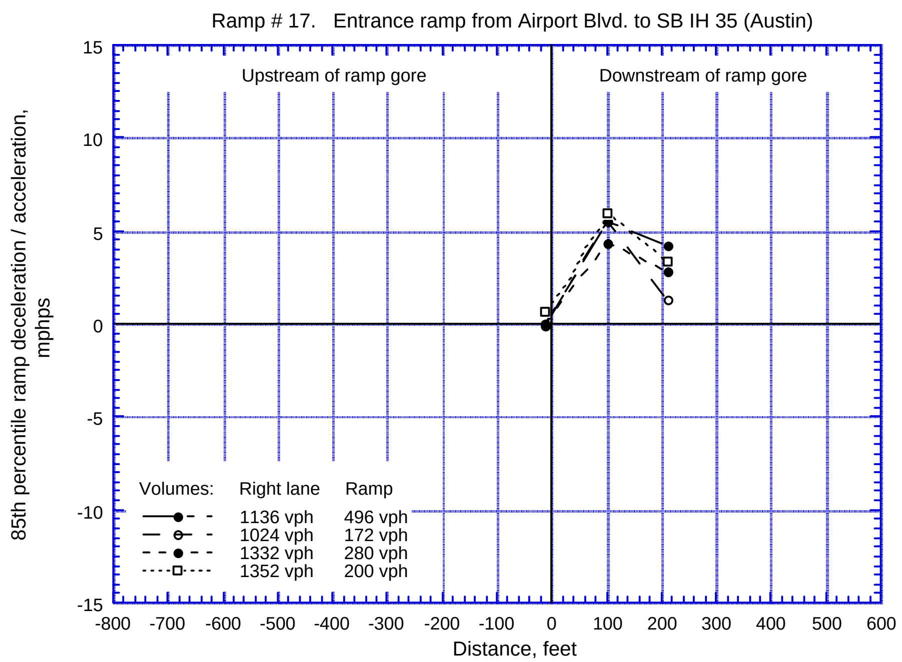

Figure D15 85th Percentile Ramp Acceleration/Deceleration, Ramp #17
Entrance Ramp from Airport Blvd. to SB IH 35, Austin

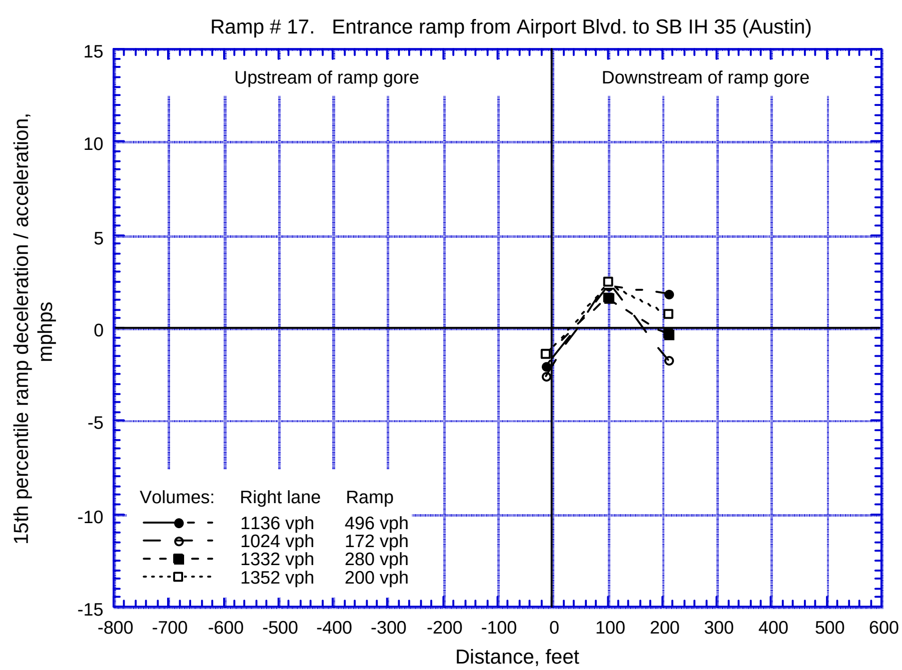

Figure D16 15th Percentile Ramp Acceleration/Deceleration, Ramp #17
Entrance Ramp from Airport Blvd. to SB IH 35, Austin

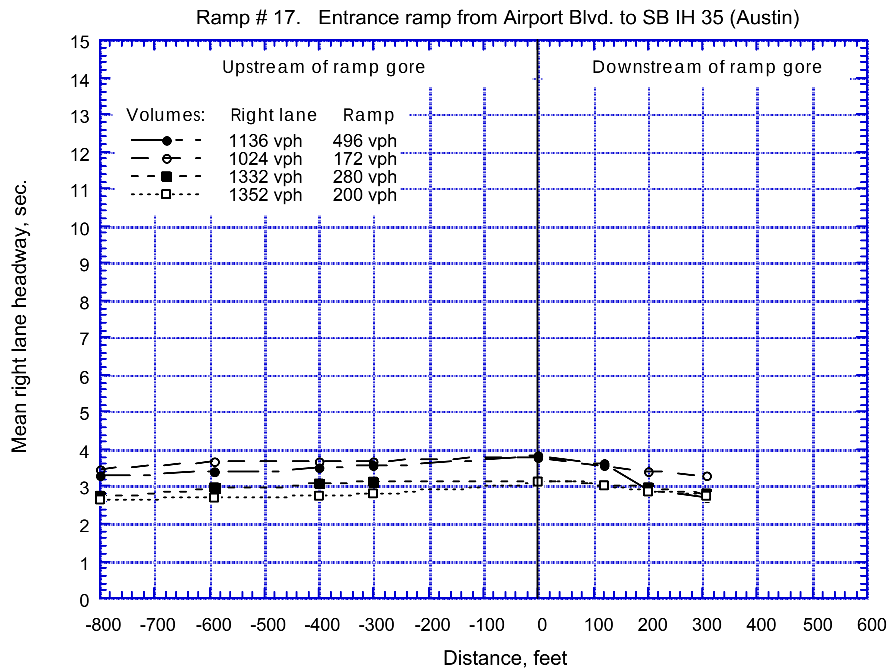

Figure D17 Mean Time Headway Freeway Right Lane, Ramp #17
Entrance Ramp from Airport Blvd. to SB IH 35, Austin

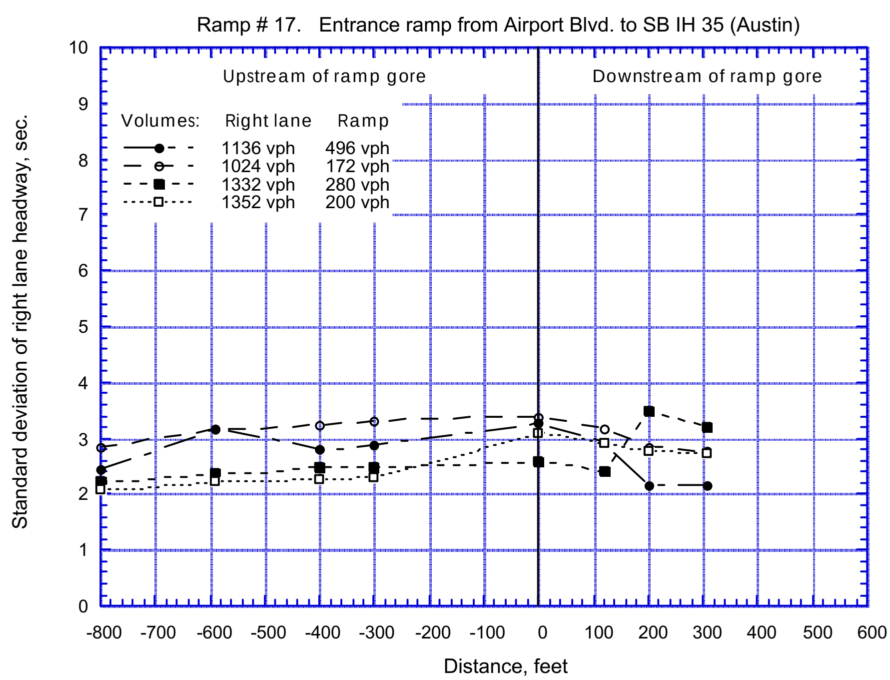

Figure D18 Standard Deviation of Time Headway Freeway Right Lane,
Ramp #17 Entrance Ramp from Airport Blvd. to SB IH 35, Austin

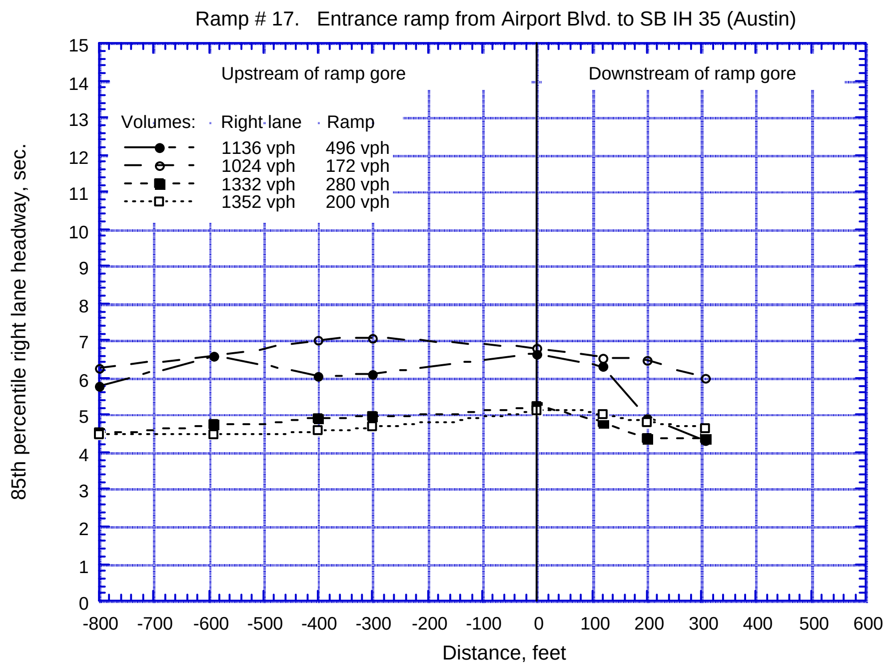

Figure D19 85th Percentile Time Headway Freeway Right Lane, Ramp #17
Entrance Ramp from Airport Blvd. to SB IH 35, Austin

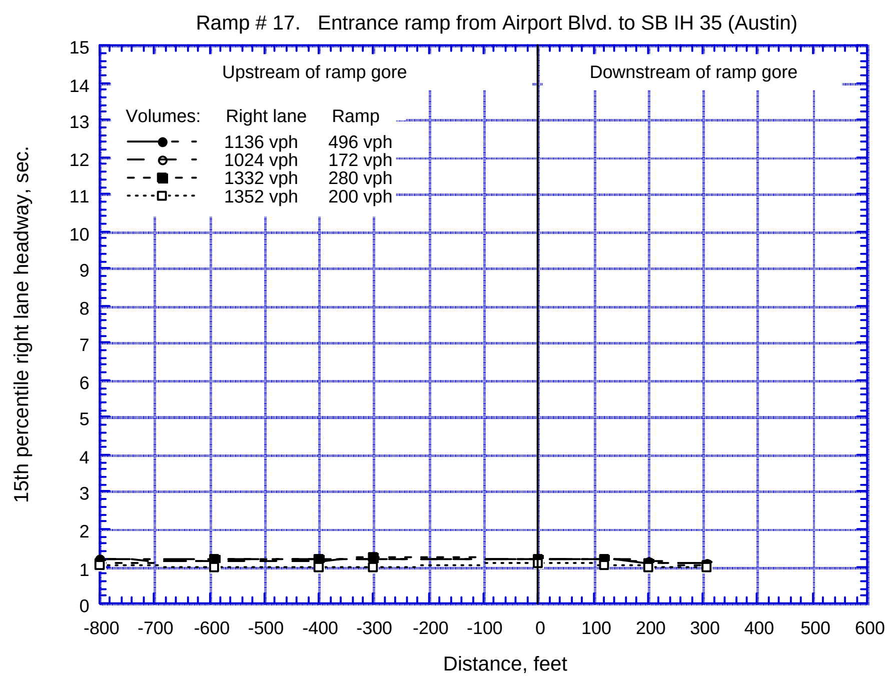

Figure D20 15th Percentile Time Headway Freeway Right Lane, Ramp #17
Entrance Ramp from Airport Blvd. to SB IH 35, Austin

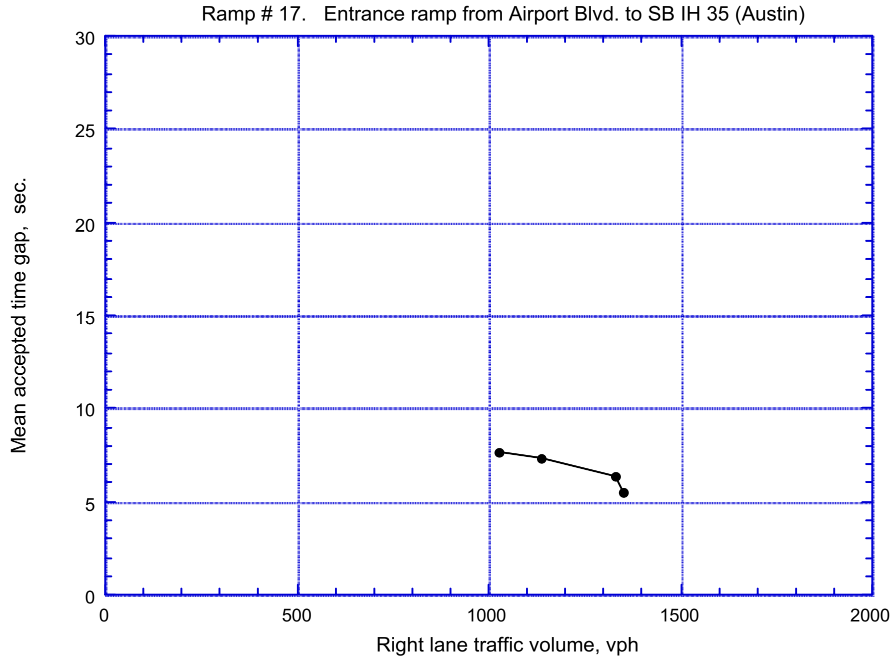

Figure D21 Mean Freeway Time Gap Accepted by Ramp Vehicles Versus Right Freeway Lane Traffic Volume, Ramp #17 Entrance Ramp from Airport Blvd. to SB IH 35, Austin

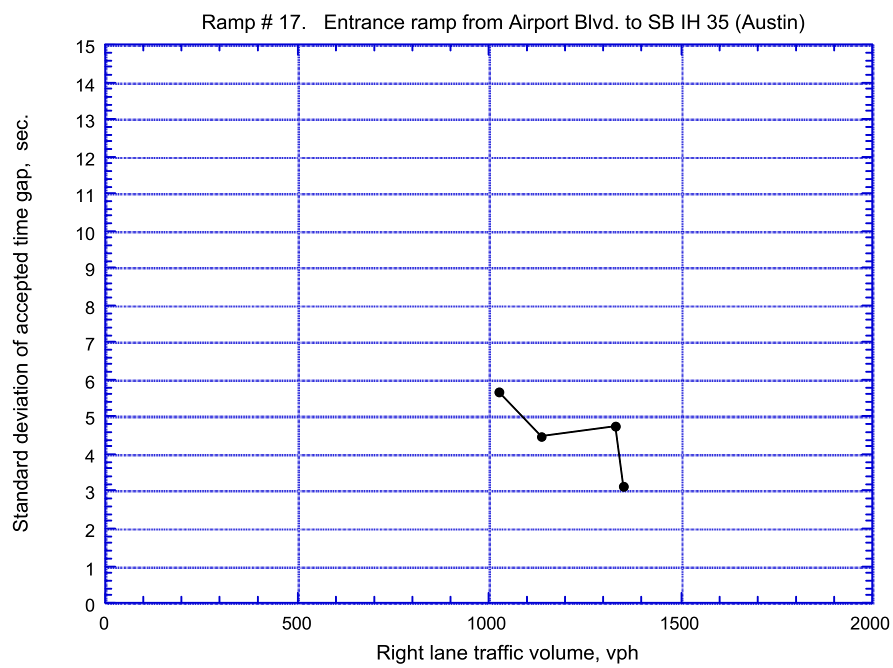

Figure D22 Standard Deviation of Freeway Time Gap Accepted by Ramp Vehicles, Ramp #17 Entrance Ramp from Airport Blvd. to SB IH 35, Austin

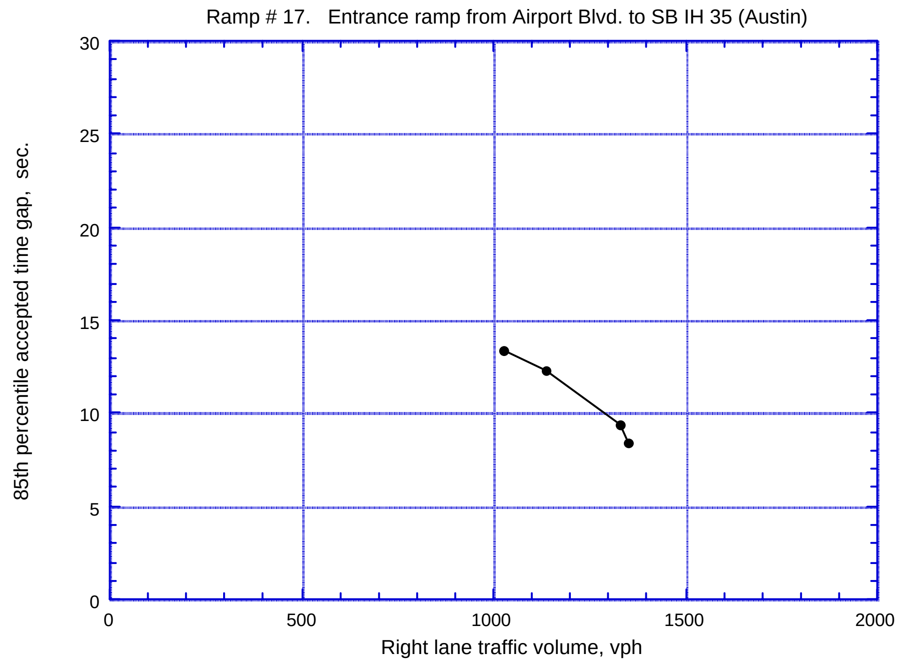

Figure D23 85th Percentile Freeway Time Gap Accepted by Ramp Vehicles, Ramp #17 Entrance Ramp from Airport Blvd. to SB IH 35, Austin

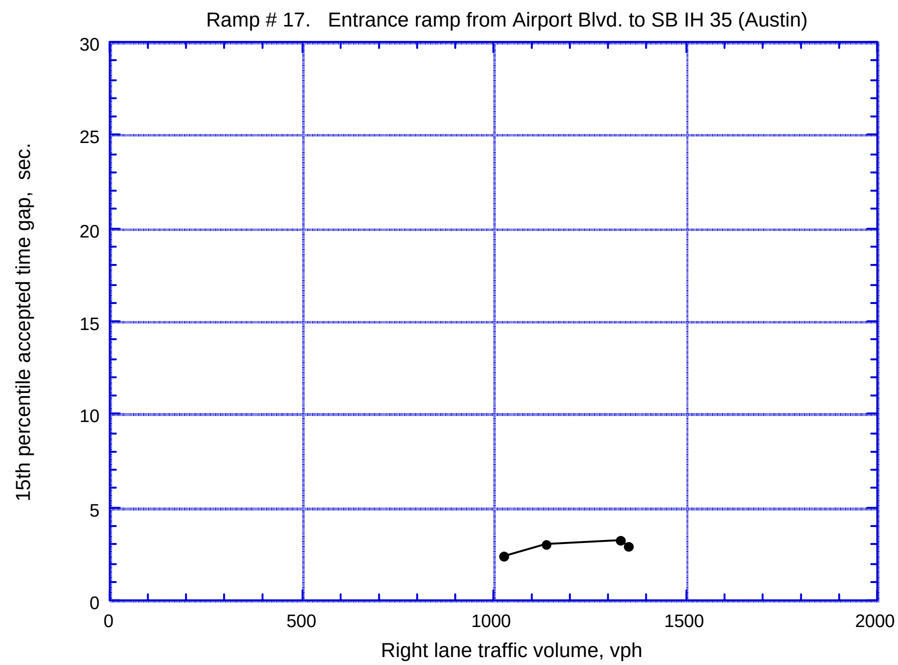

Figure D24 15th Percentile Freeway Time Gap Accepted by Ramp Vehicles, Ramp #17 Entrance Ramp from Airport Blvd. to SB IH 35, Austin

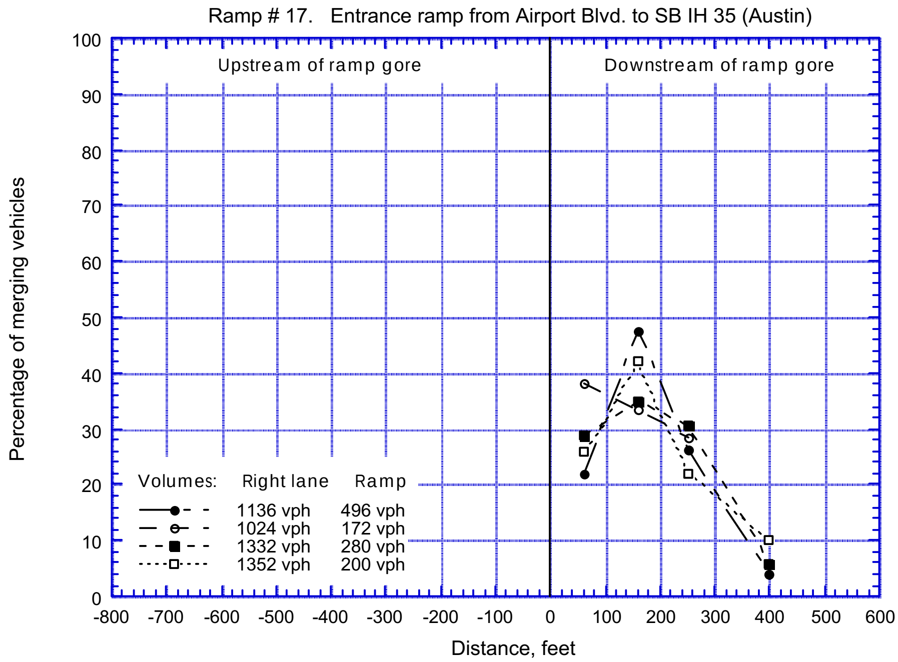

Figure D25 Ramp Vehicle Merging Location Percentage, Ramp #17
Entrance Ramp from Airport Blvd. to SB IH 35, Austin

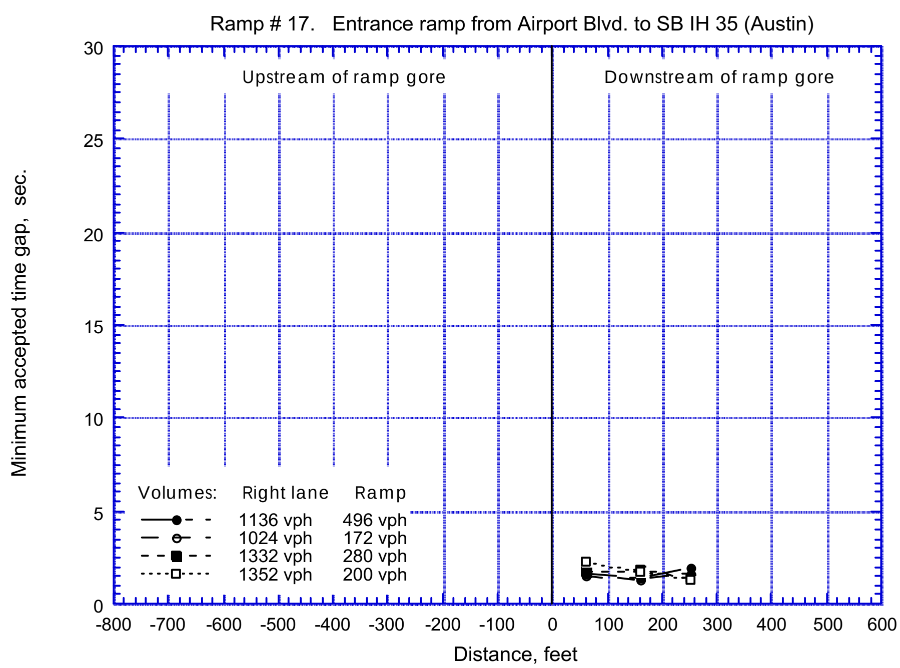

Figure D26 Minimum Time Gap Accepted by Ramp Vehicles, Ramp #17
Entrance Ramp from Airport Blvd. to SB IH 35, Austin

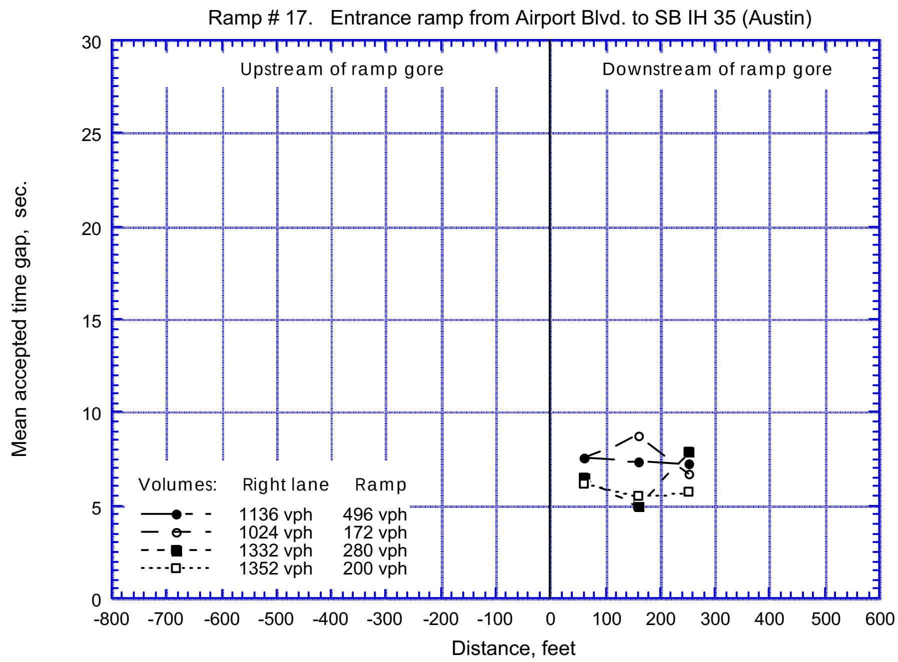

Figure D27 Mean Freeway Time Gap Accepted by Ramp Vehicles Versus Merging Location, Ramp #17 Entrance Ramp from Airport Blvd. to SB IH 35, Austin

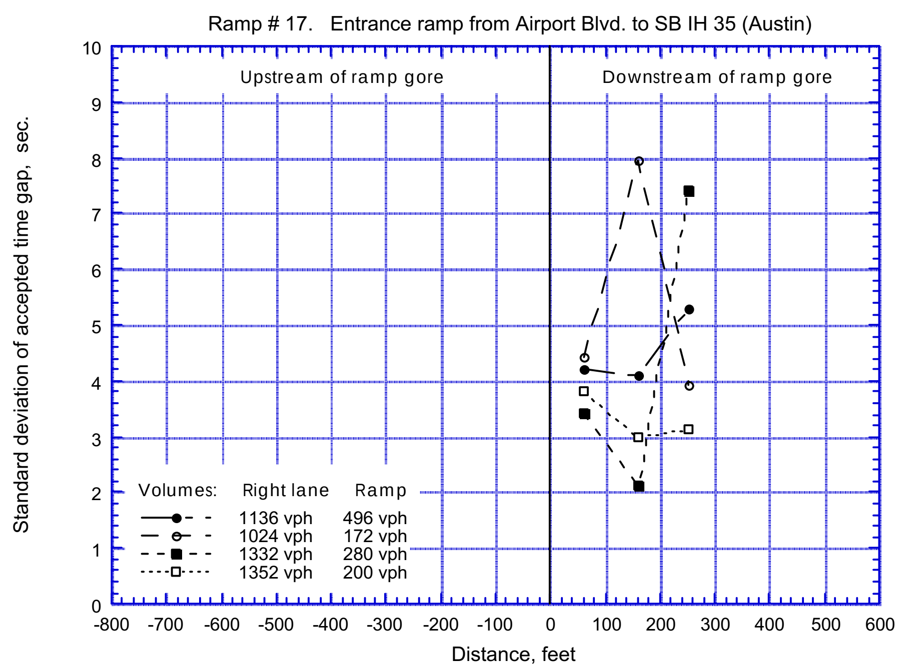

Figure D28 Standard Deviation of Freeway Time Gap Accepted by Ramp Vehicles Versus Merging Location, Ramp #17 Entrance Ramp from Airport Blvd. to SB IH 35, Austin

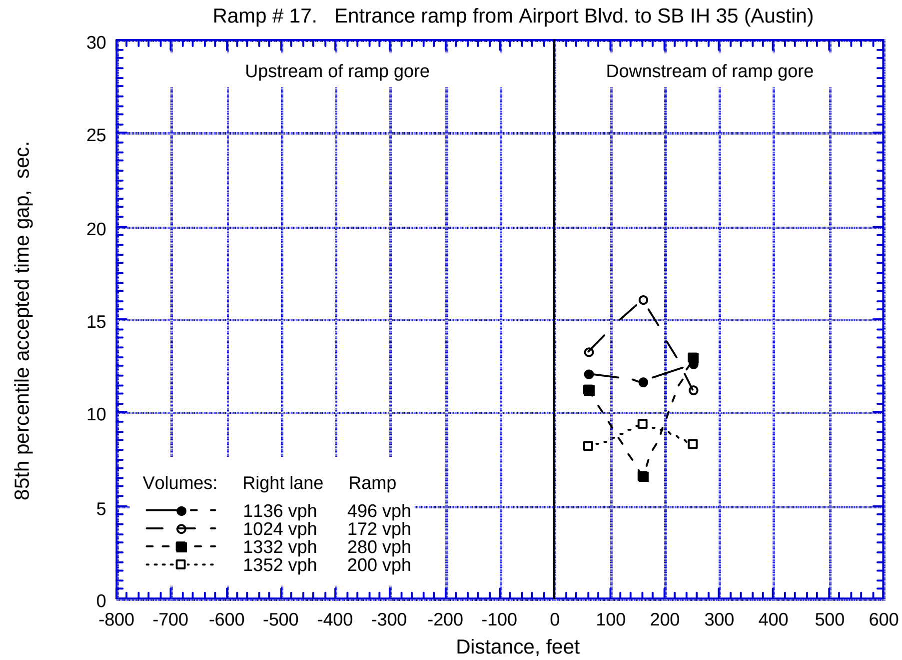

Figure D29 85th Percentile Freeway Time Gap Accepted by Ramp Vehicles Versus Merging Location, Ramp #17 Entrance Ramp from Airport Blvd. to SB IH 35, Austin

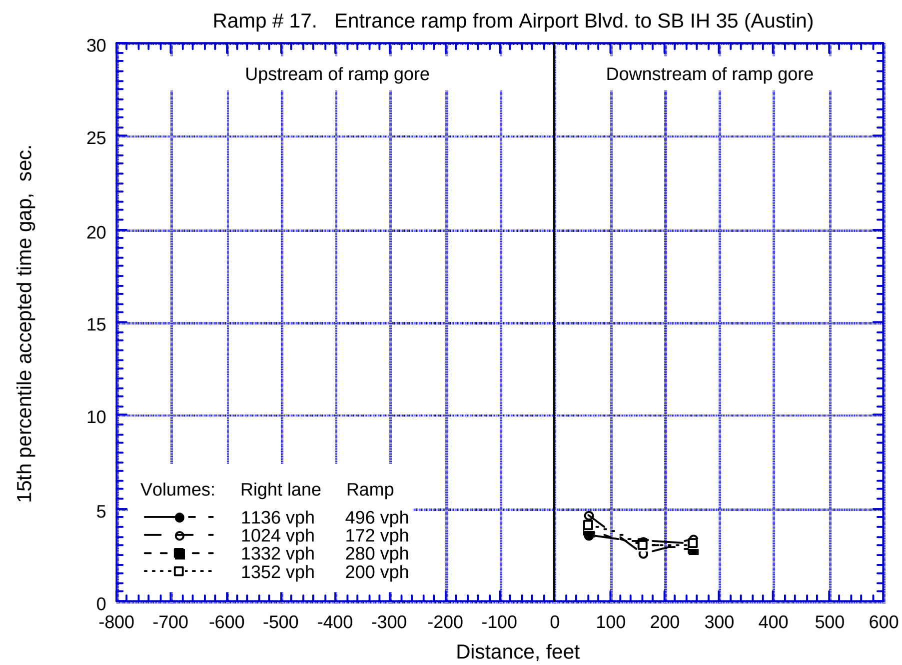

Figure D30 15th Percentile Freeway Time Gap Accepted by Ramp Vehicles Versus Merging Location, Ramp #17 Entrance Ramp from Airport Blvd. to SB IH 35, Austin

APPENDIX E

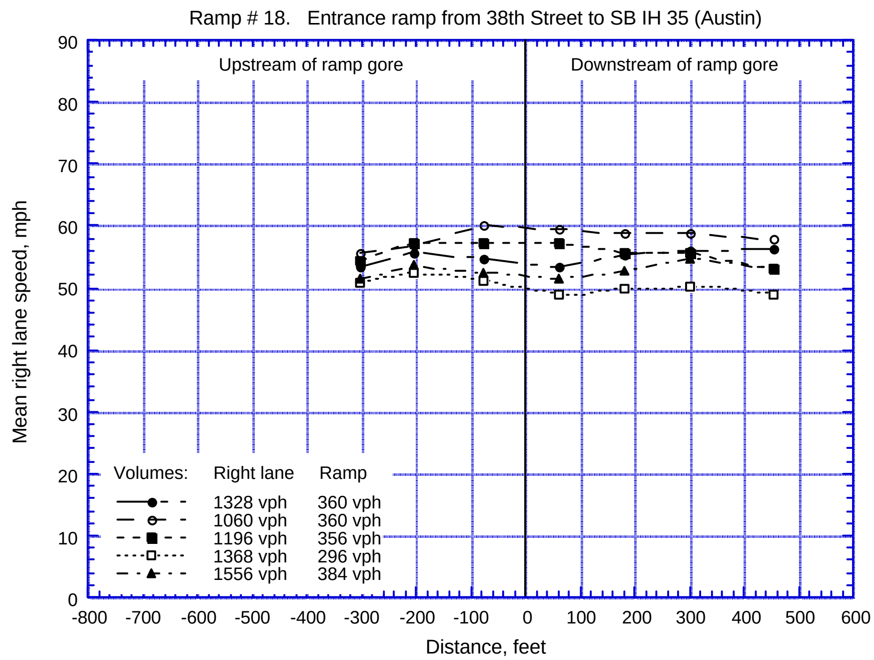

Figure E01 Mean Freeway Right Lane Speed, Ramp #18, Entrance Ramp from 38th Street to SB IH 35, Austin

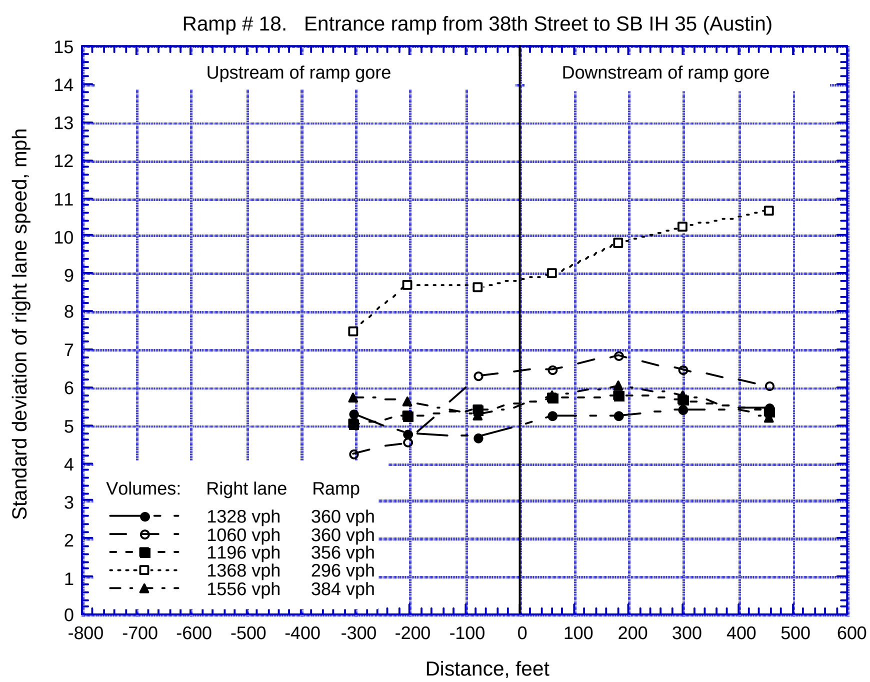

Figure E02 Standard Deviation of Freeway Right Lane Speed, Ramp #18 Entrance Ramp from 38th Street to SB IH 35, Austin

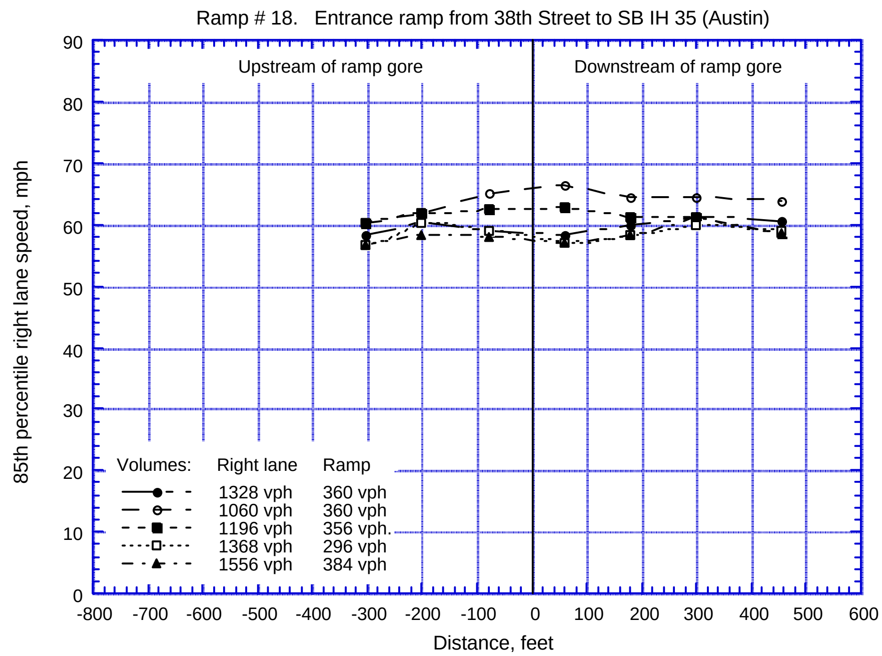

Figure E03 85th Percentile Freeway Right Lane Speed, Ramp #18
Entrance Ramp from 38th Street to SB IH 35, Austin

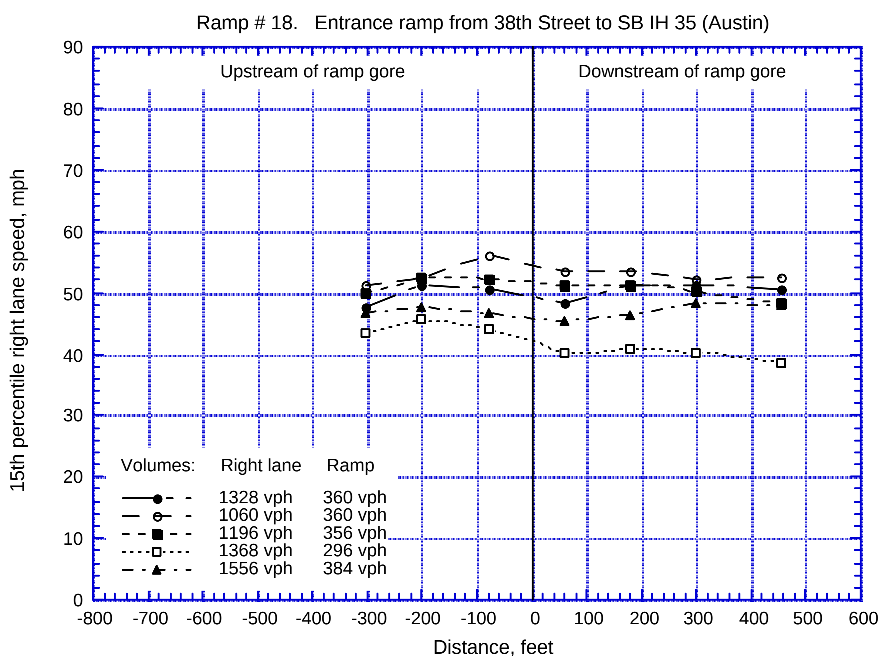

Figure E04 15th Percentile Freeway Right Lane Speed, Ramp #18
Entrance Ramp from 38th Street to SB IH 35, Austin

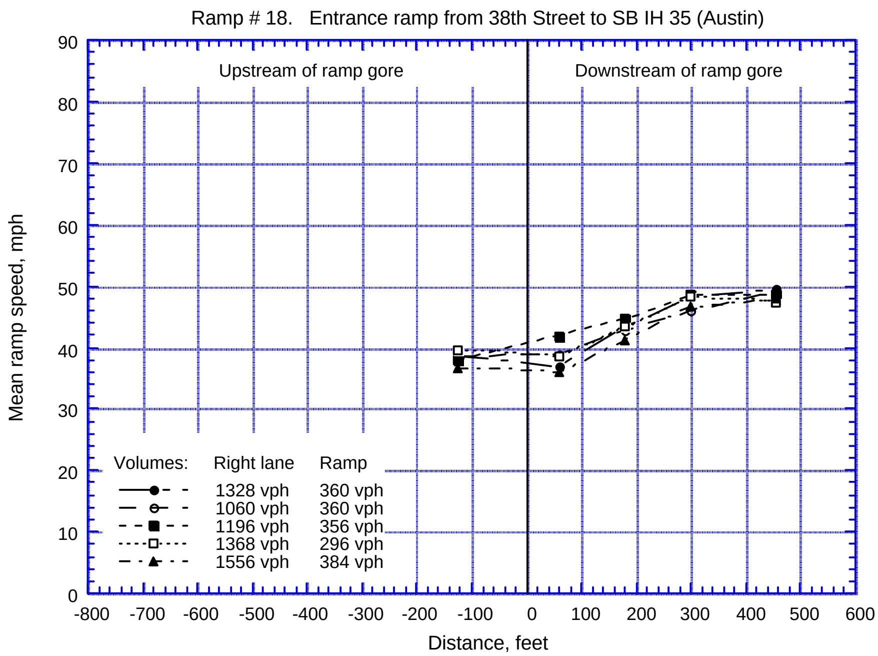

Figure E05 Mean Ramp Speed, Ramp #18 Entrance Ramp from 38th Street to SB IH 35, Austin

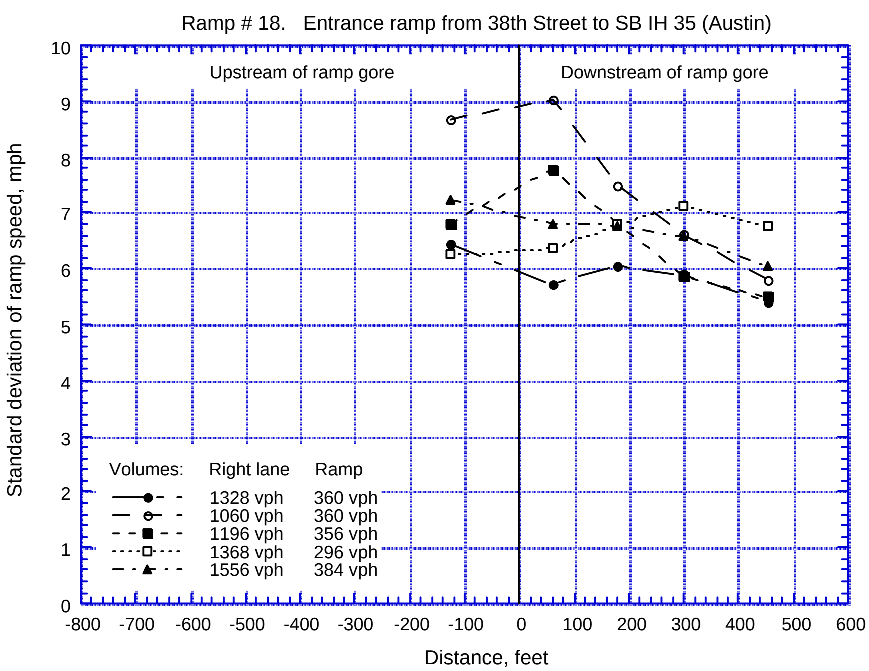

Figure E06 Standard Deviation of Ramp Speed, Ramp #18 Entrance Ramp from 38th Street to SB IH 35, Austin

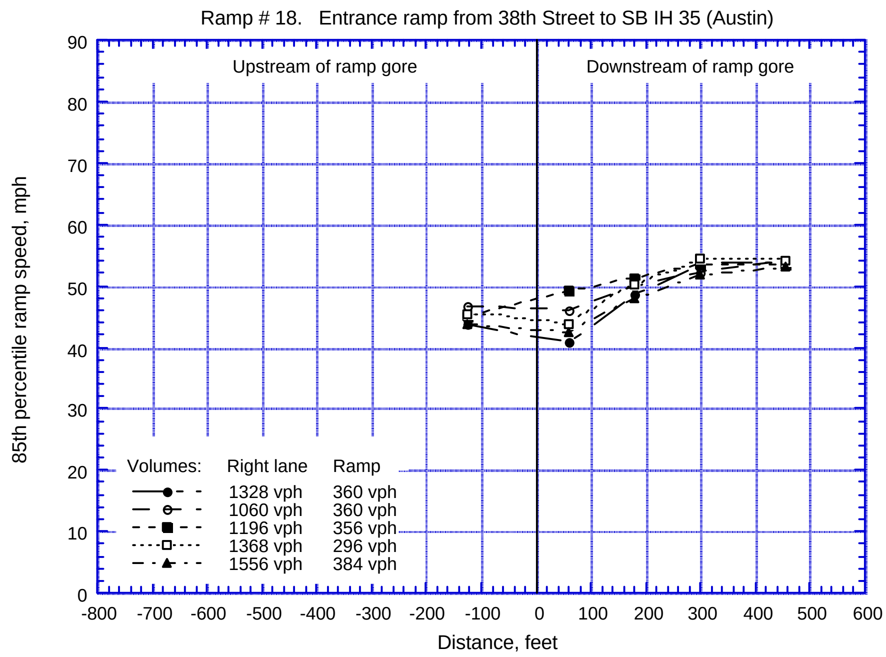

Figure E07 85th Percentile Ramp Speed, Ramp #18
Entrance Ramp from 38th Street to SB IH 35, Austin

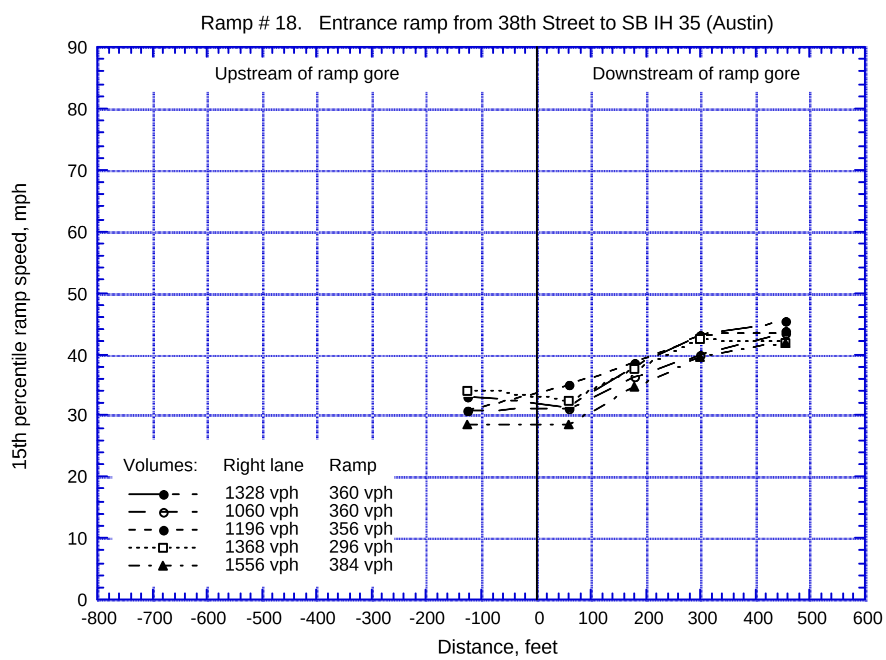

Figure E08 15th Percentile Ramp Speed, Ramp #18
Entrance Ramp from 38th Street to SB IH 35, Austin

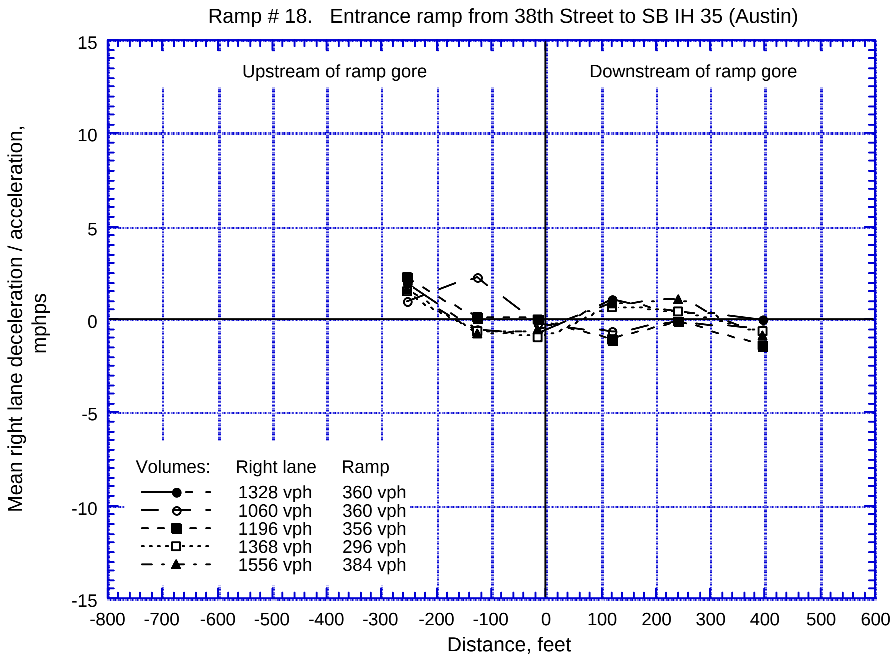

Figure E09 Mean Freeway Right Lane Acceleration/Deceleration,
Ramp #18 Entrance Ramp from 38th Street to SB IH 35, Austin

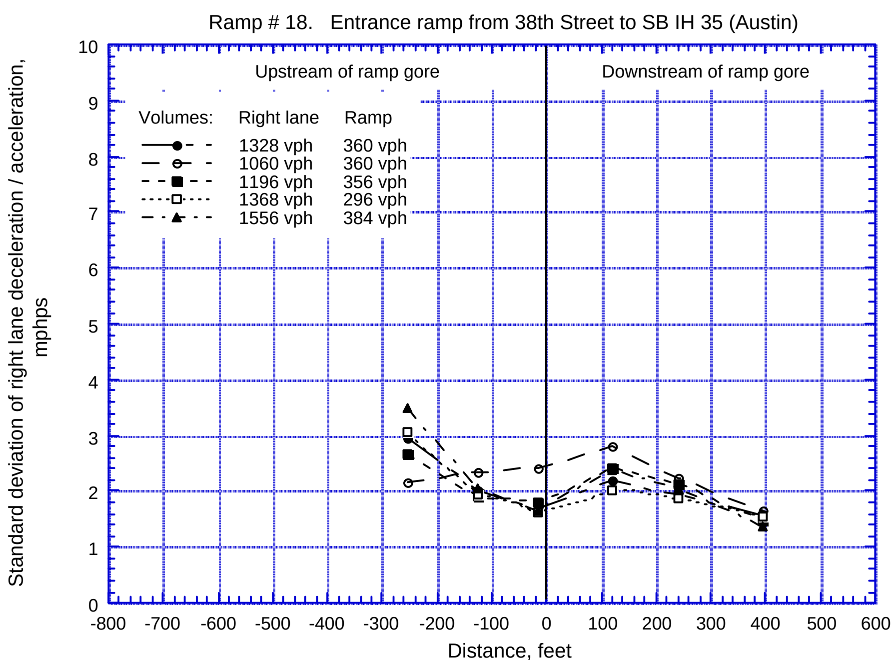

Figure E10 Standard Deviation of Freeway Right Lane Acceleration/Deceleration,
Ramp #18 Entrance Ramp from 38th Street to SB IH 35, Austin

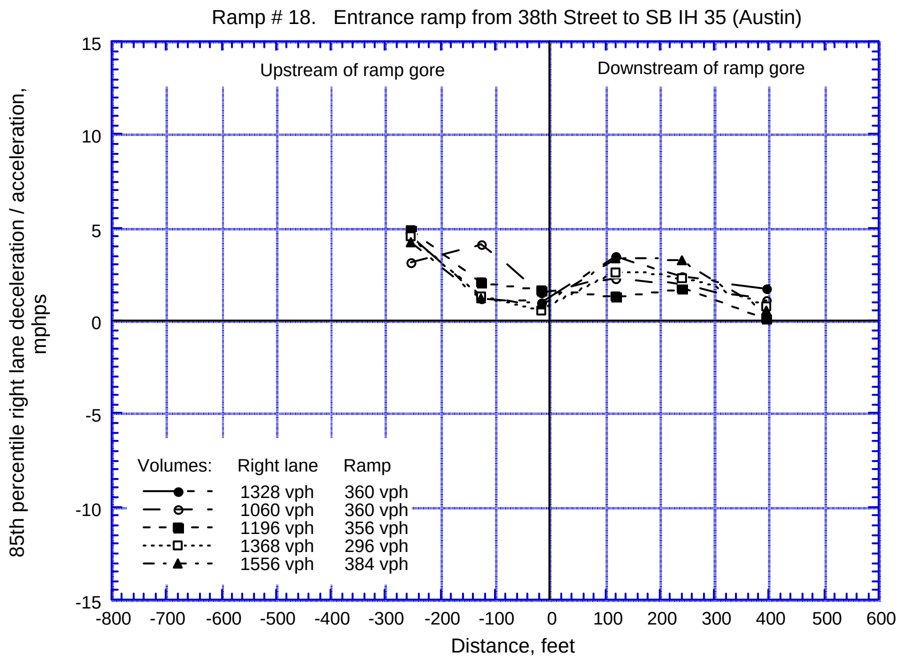

Figure E11 85th Percentile Freeway Right Lane Acceleration/Deceleration, Ramp #18 Entrance Ramp from 38th Street to SB IH 35, Austin

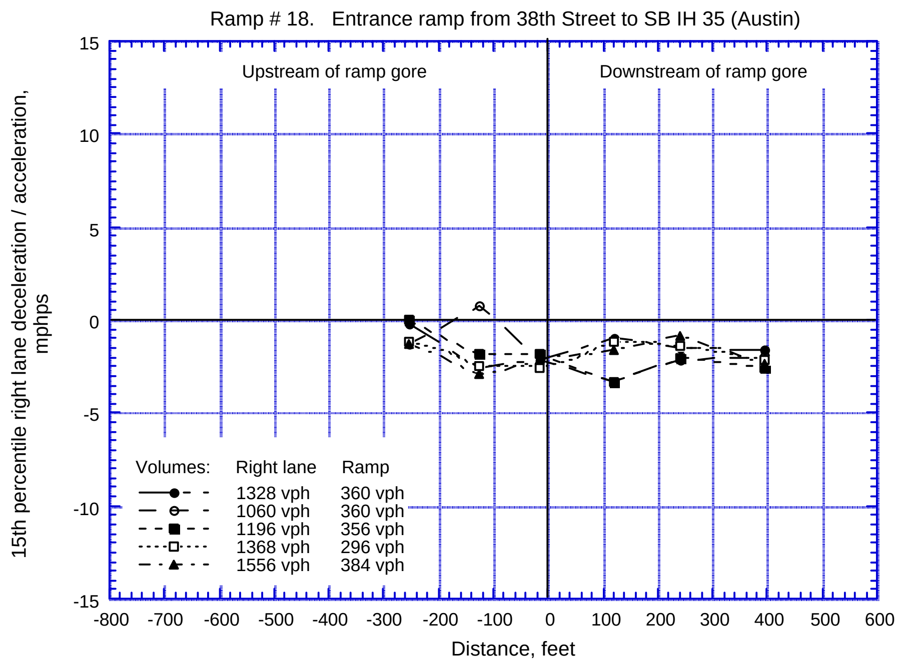

Figure E12 15th Percentile Freeway Right Lane Acceleration/Deceleration, Ramp #18 Entrance Ramp from 38th Street to SB IH 35, Austin

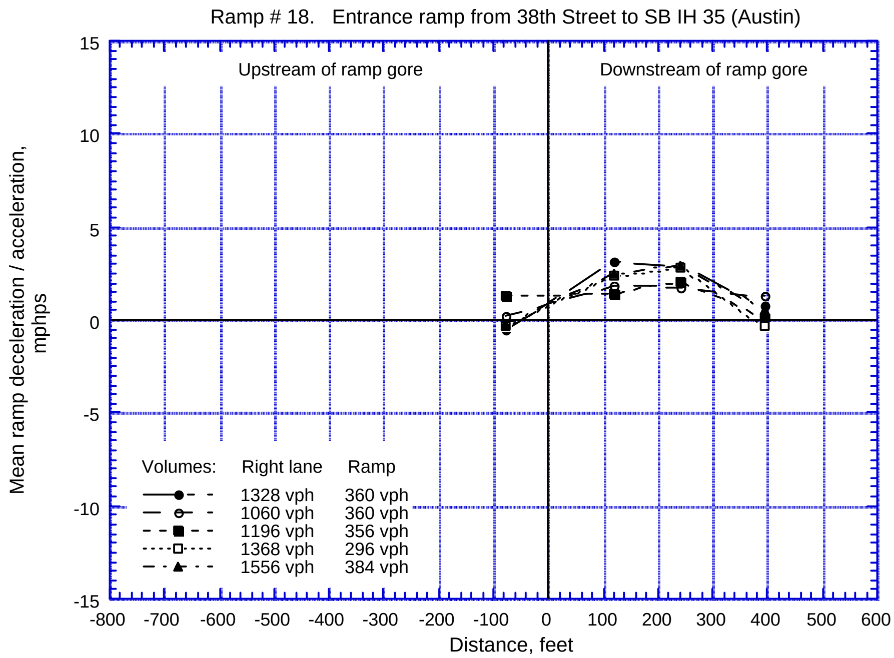

Figure E13 Mean Ramp Acceleration/Deceleration, Ramp #18
Entrance Ramp from 38th Street to SB IH 35, Austin

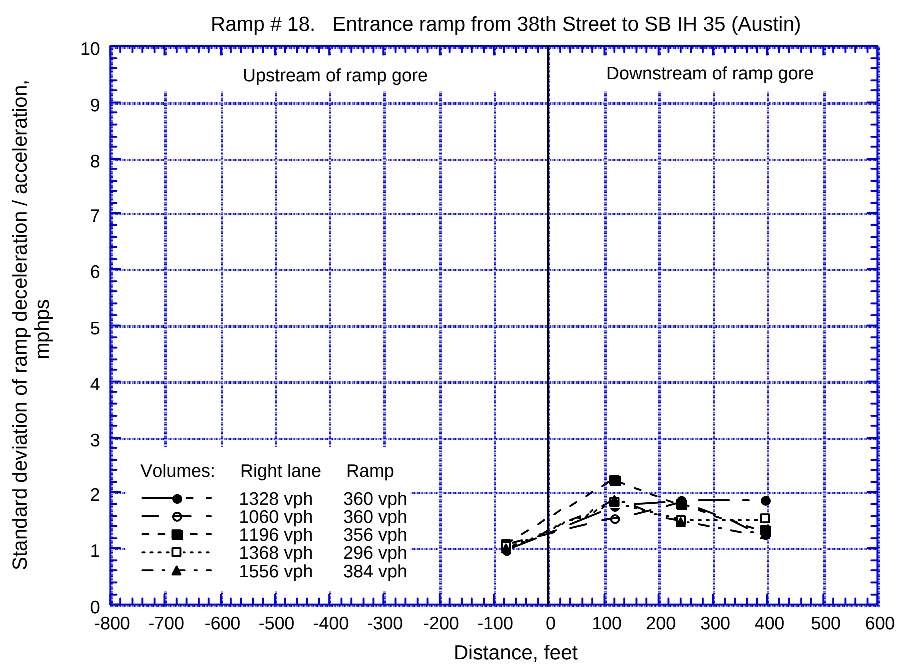

Figure E14 Standard Deviation of Ramp Acceleration/Deceleration,
Ramp #18 Entrance Ramp from 38th Street to SB IH 35, Austin

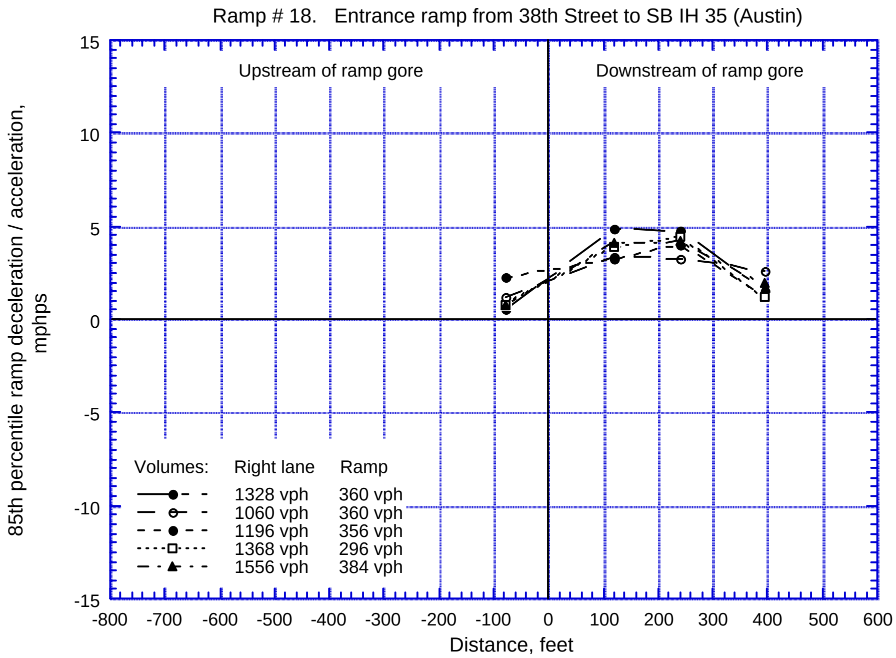

Figure E15 85th Percentile Ramp Acceleration/Deceleration, Ramp #18
Entrance Ramp from 38th Street to SB IH 35, Austin

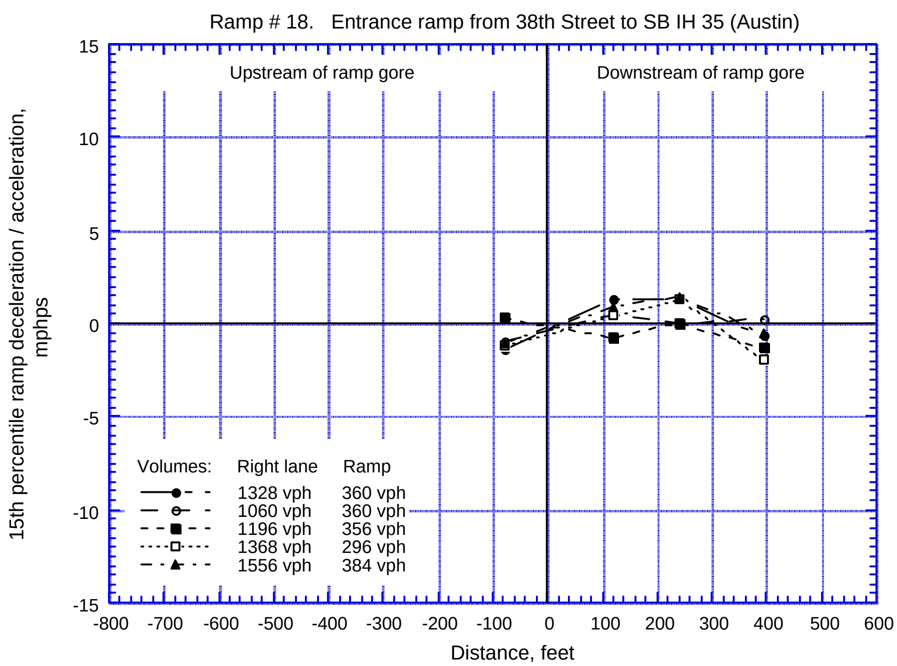

Figure E16 15th Percentile Ramp Acceleration/Deceleration, Ramp #18
Entrance Ramp from 38th Street to SB IH 35, Austin

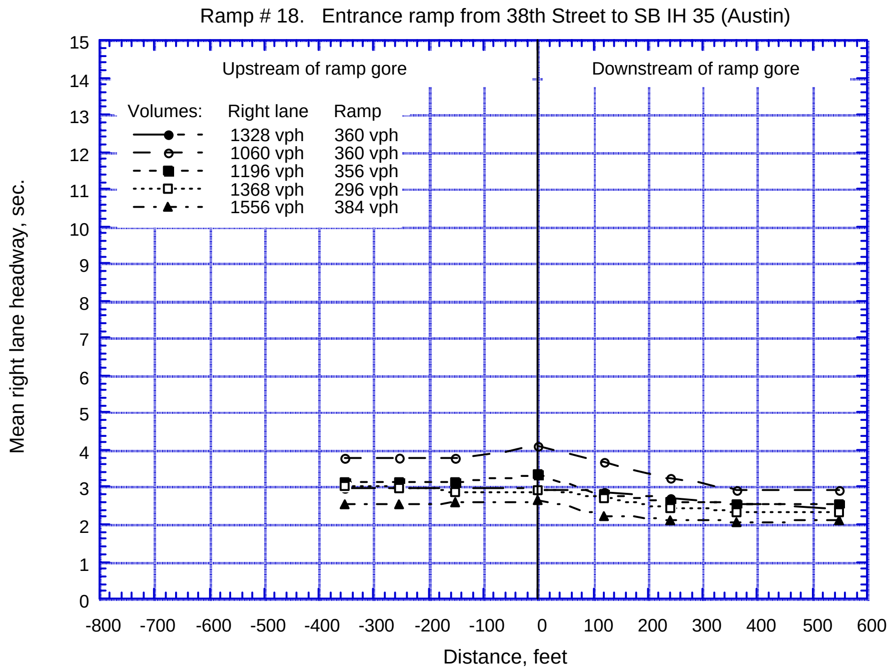

Figure E17 Mean Time Headway Freeway Right Lane, Ramp #18
Entrance Ramp from 38th Street to SB IH 35, Austin

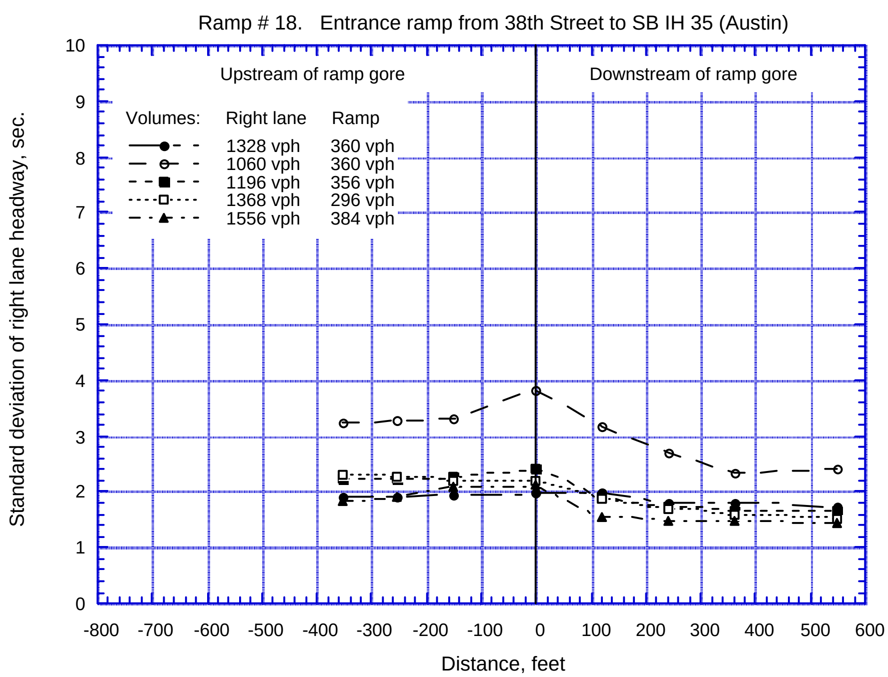

Figure E18 Standard Deviation of Time Headway Freeway Right Lane,
Ramp #18 Entrance Ramp from 38th Street to SB IH 35, Austin

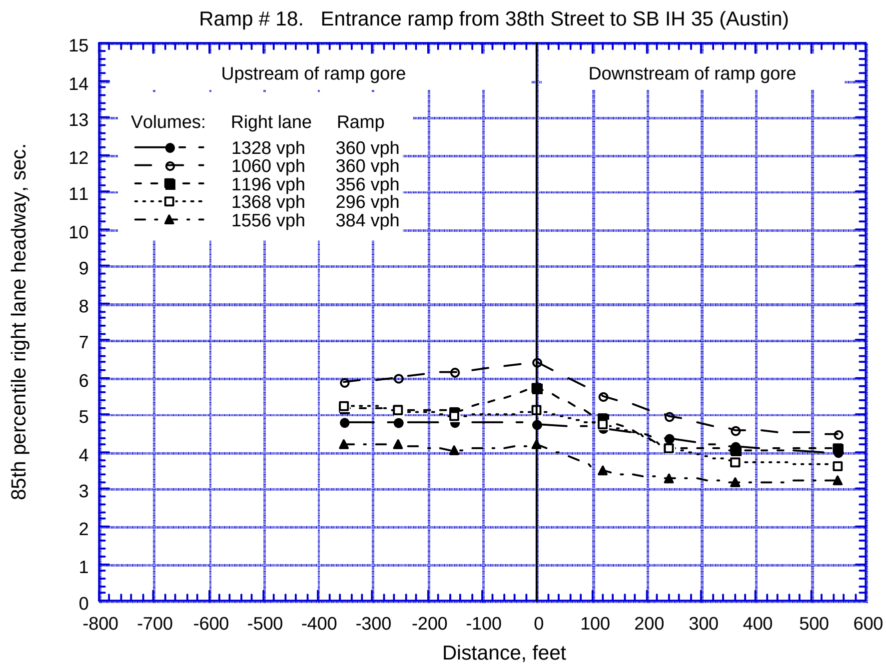

Figure E19 85th Percentile Time Headway Freeway Right Lane,
Ramp #18 Entrance Ramp from 38th Street to SB IH 35, Austin

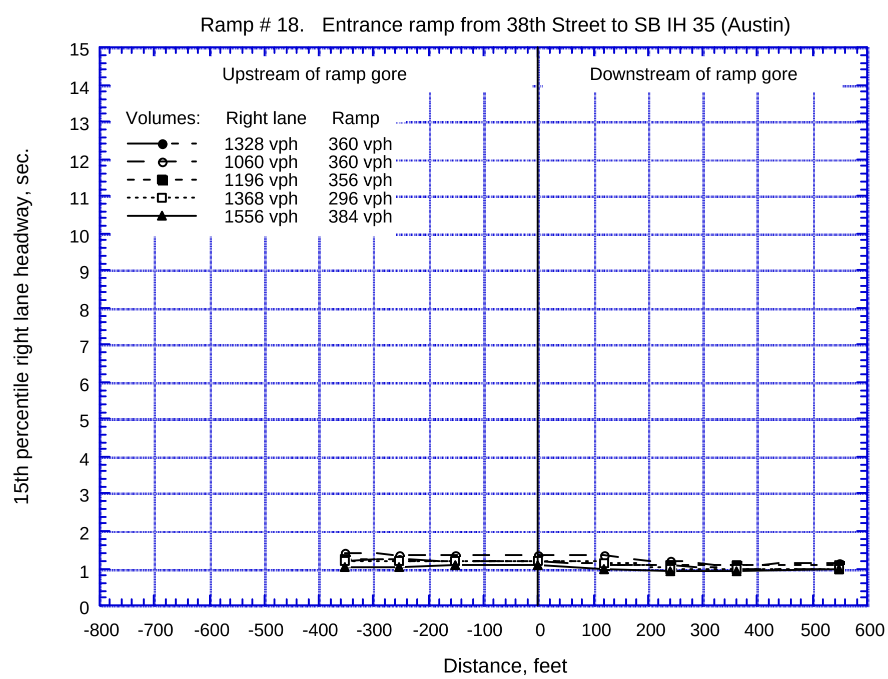

Figure E20 15th Percentile Time Headway Freeway Right Lane,
Ramp #18 Entrance Ramp from 38th Street to SB IH 35, Austin

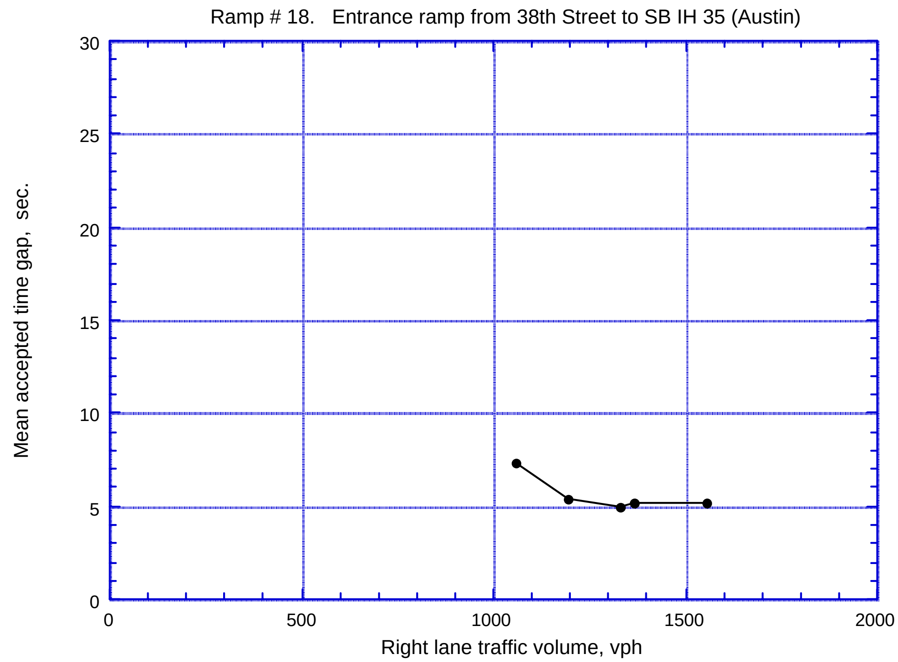

Figure E21 Mean Freeway Time Gap Accepted by Ramp Vehicles Versus Right Freeway Lane Traffic Volume, Ramp #18 Entrance Ramp from 38th Street to SB IH 35, Austin

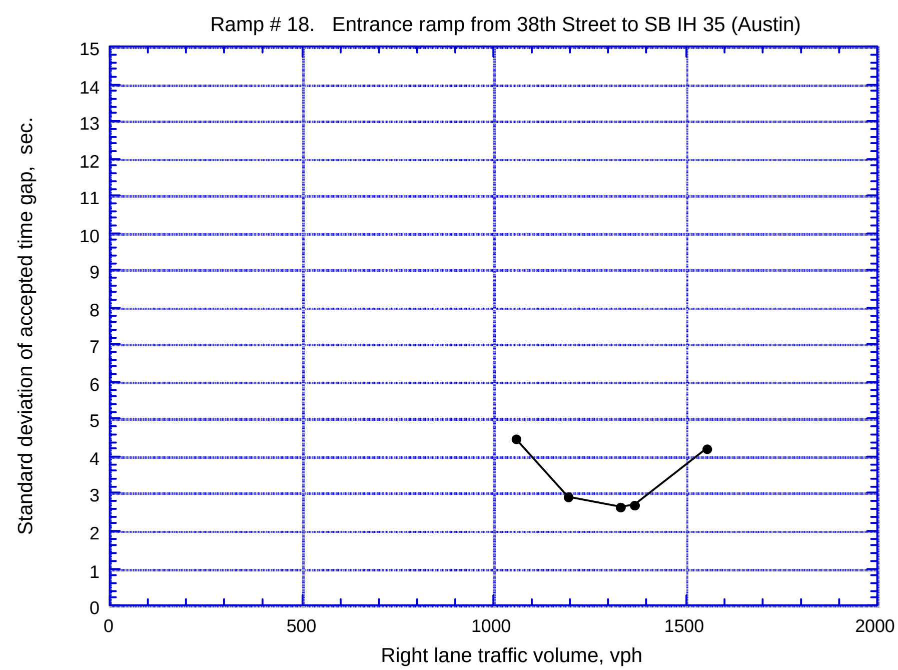

Figure E22 Standard Deviation of Freeway Time Gap Accepted by Ramp Vehicles, Ramp #18 Entrance Ramp from 38th Street to SB IH 35, Austin

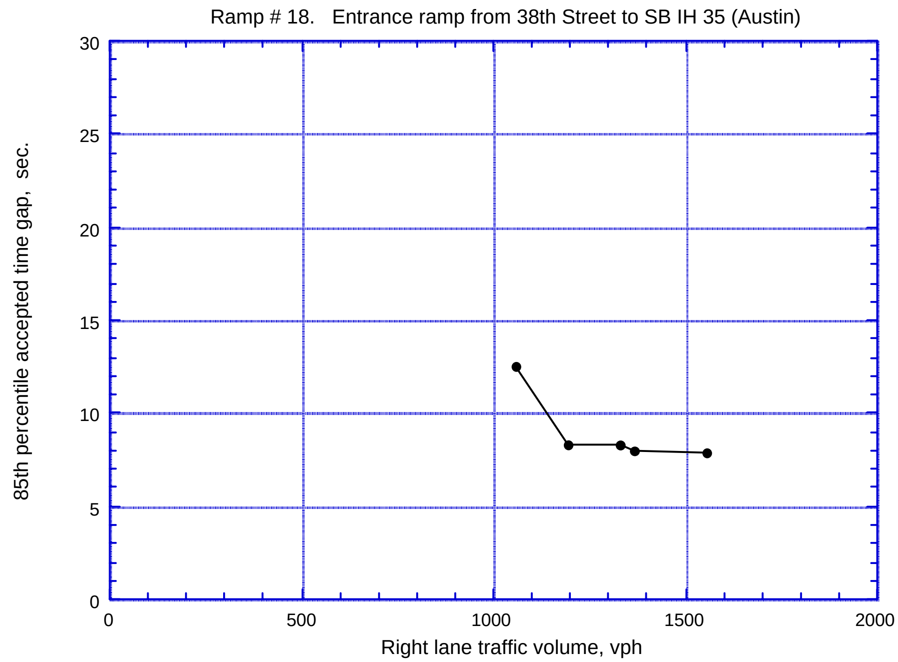

Figure E23 85th Percentile Freeway Time Gap Accepted by Ramp Vehicles, Ramp #18 Entrance Ramp from 38th Street to SB IH 35, Austin

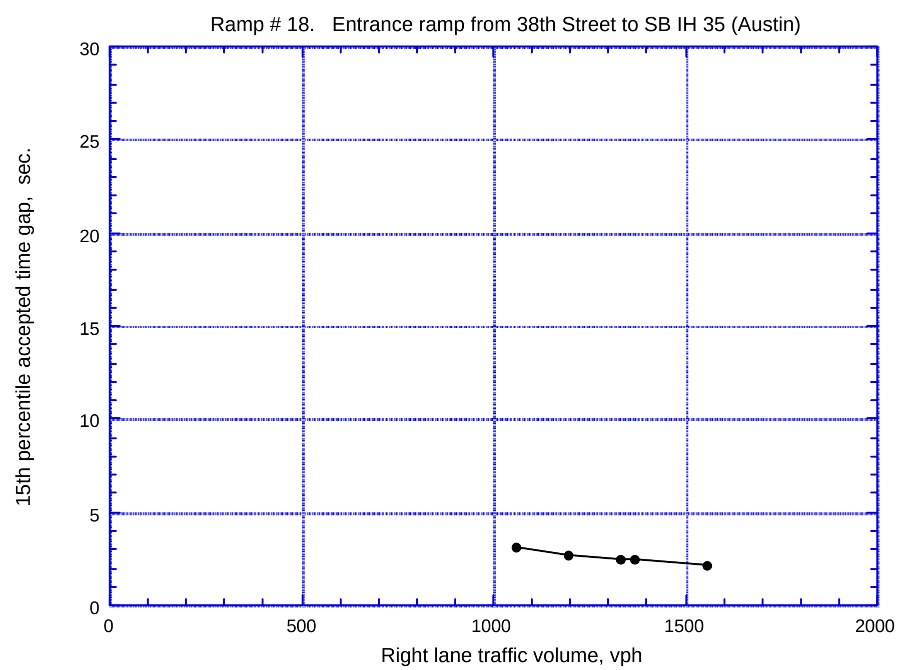

Figure E24 15th Percentile Freeway Time Gap Accepted by Ramp Vehicles, Ramp #18 Entrance Ramp from 38th Street to SB IH 35, Austin

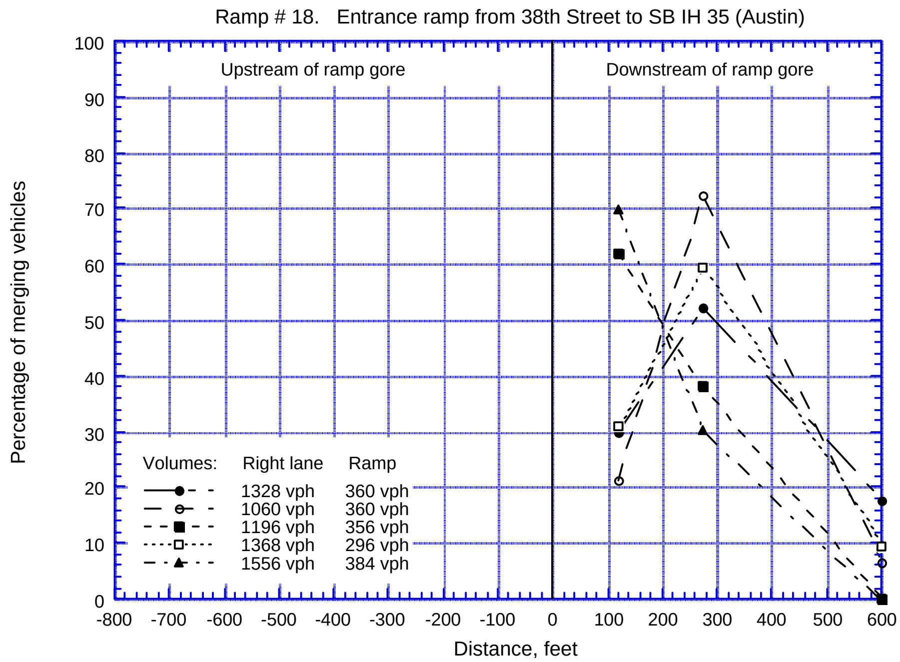

Figure E25 Ramp Vehicle Merging Location Percentage, Ramp #18
Entrance Ramp from 38th Street to SB IH 35, Austin

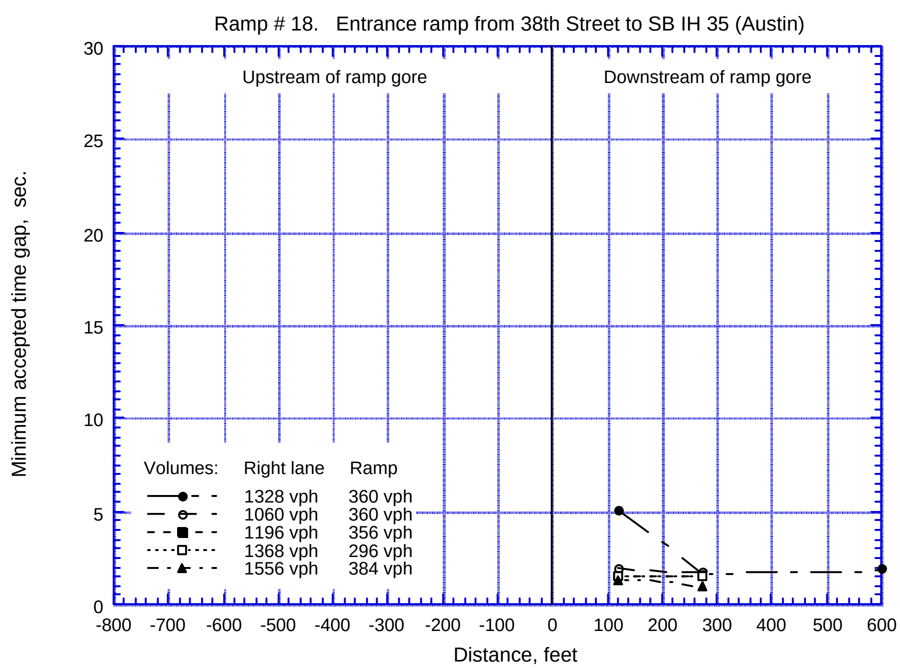

Figure E26 Minimum Time Gap Accepted by Ramp Vehicles,
Ramp #18 Entrance Ramp from 38th Street to SB IH 35, Austin

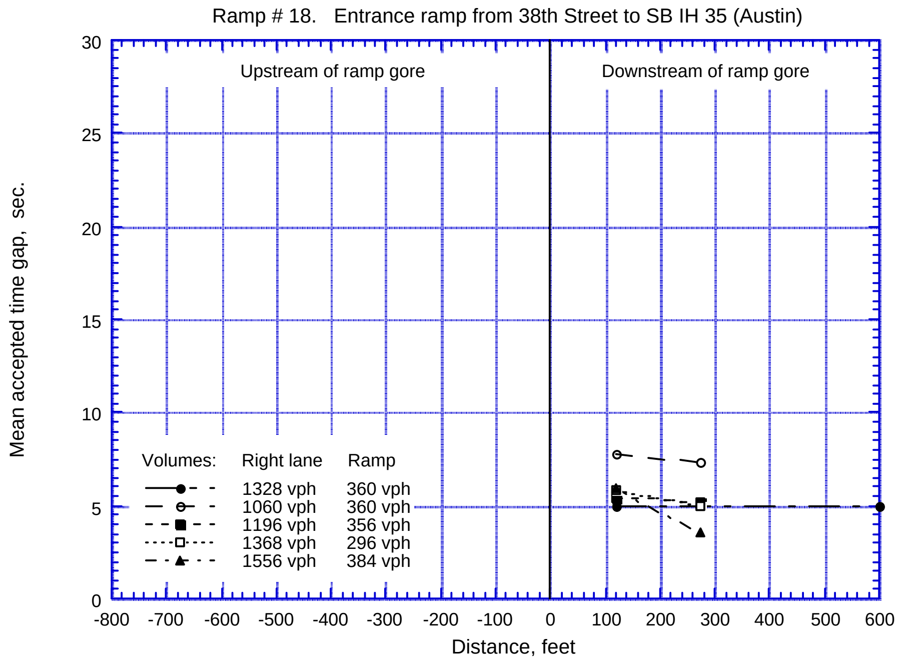

Figure E27 Mean Freeway Time Gap Accepted by Ramp Vehicles Versus Merging Location, Ramp #18 Entrance Ramp from 38th Street to SB IH 35, Austin

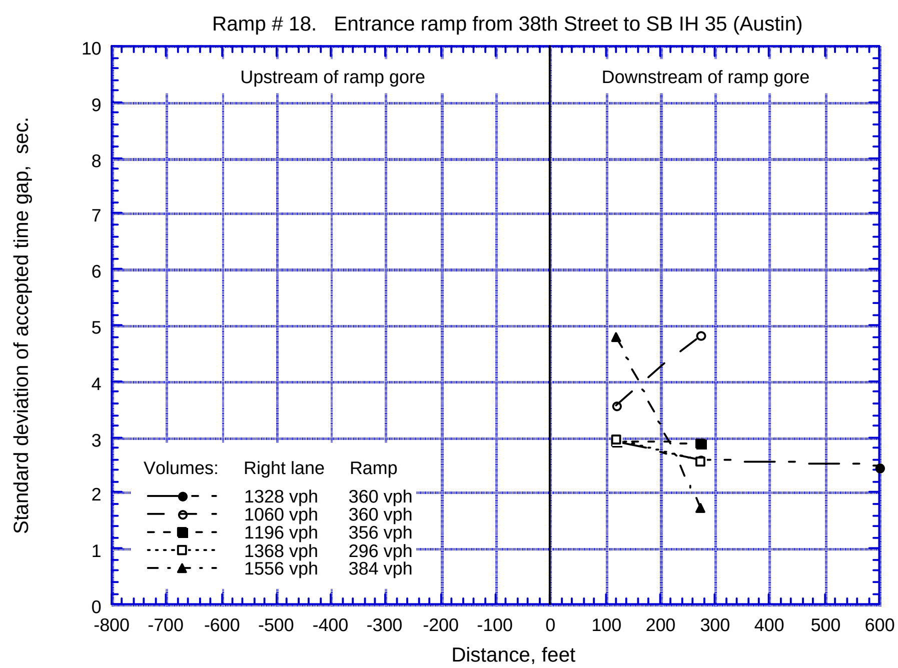

Figure E28 Standard Deviation of Freeway Time Gap Accepted by Ramp Vehicles Versus Merging Location, Ramp #18 Entrance Ramp from 38th Street to SB IH 35, Austin

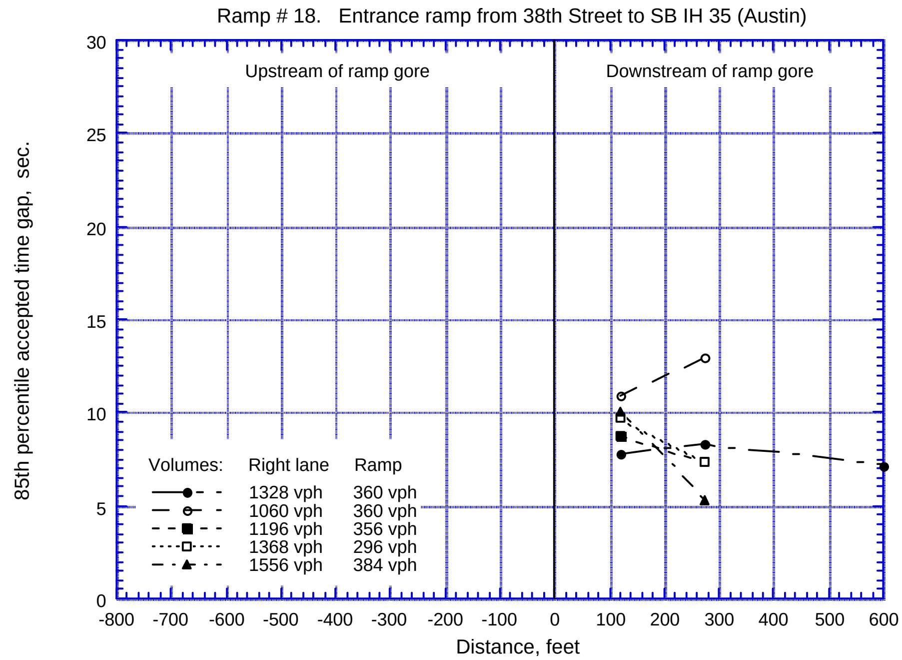

Figure E29 85th Percentile Freeway Time Gap Accepted by Ramp Vehicles Versus Merging Location, Ramp #18 Entrance Ramp from 38th Street to SB IH 35, Austin

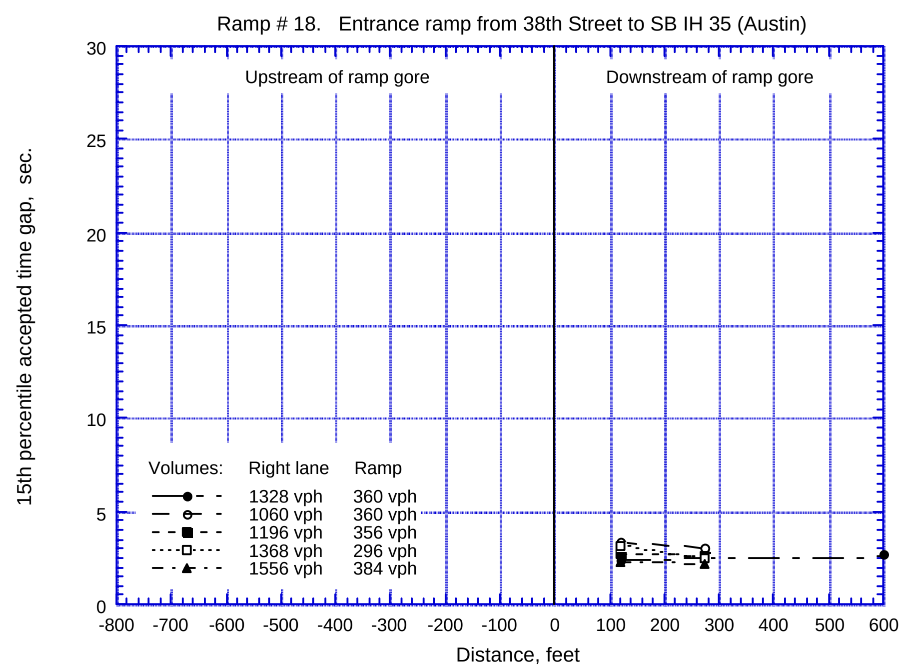

Figure E30 15th Percentile Freeway Time Gap Accepted by Ramp Vehicles Versus Merging Location, Ramp #18 Entrance Ramp from 38th Street to SB IH 35, Austin

APPENDIX F

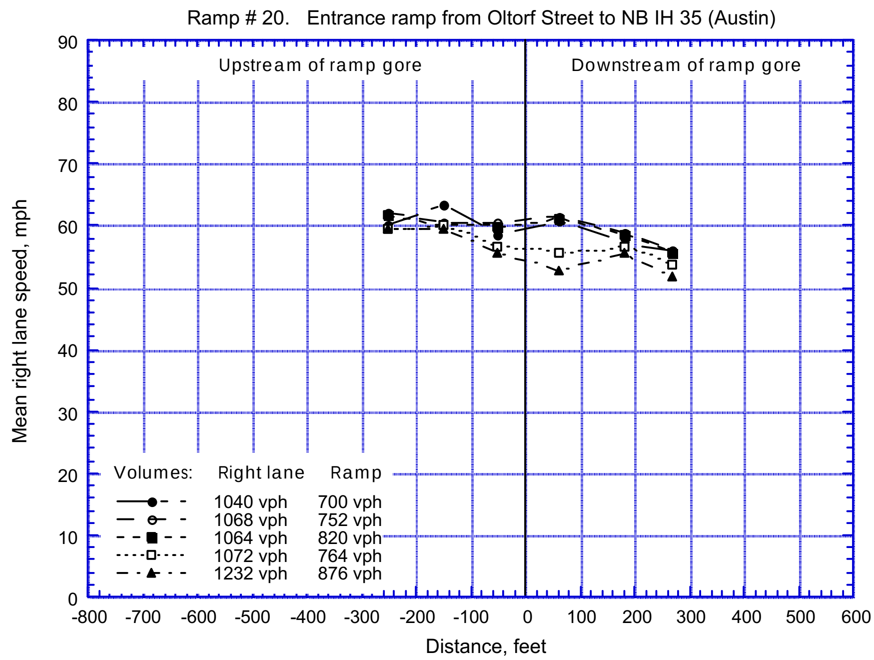

Figure F01 Mean Freeway Right Lane Speed, Ramp #20
Entrance Ramp from Oltorf Street to NB IH 35, Austin

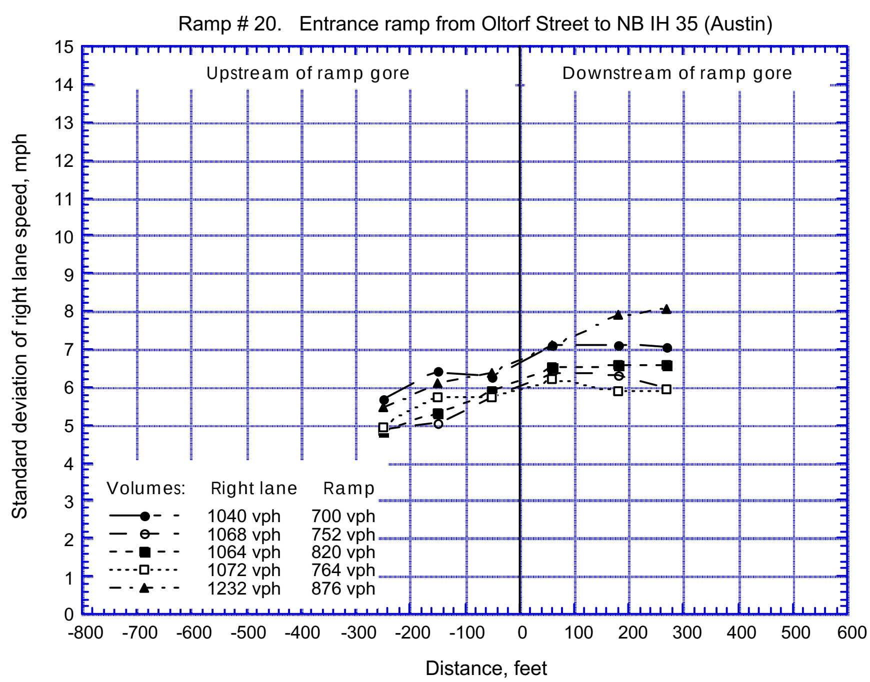

Figure F02 Standard Deviation of Freeway Right Lane Speed,
Ramp #20 Entrance Ramp from Oltorf Street to NB IH 35, Austin

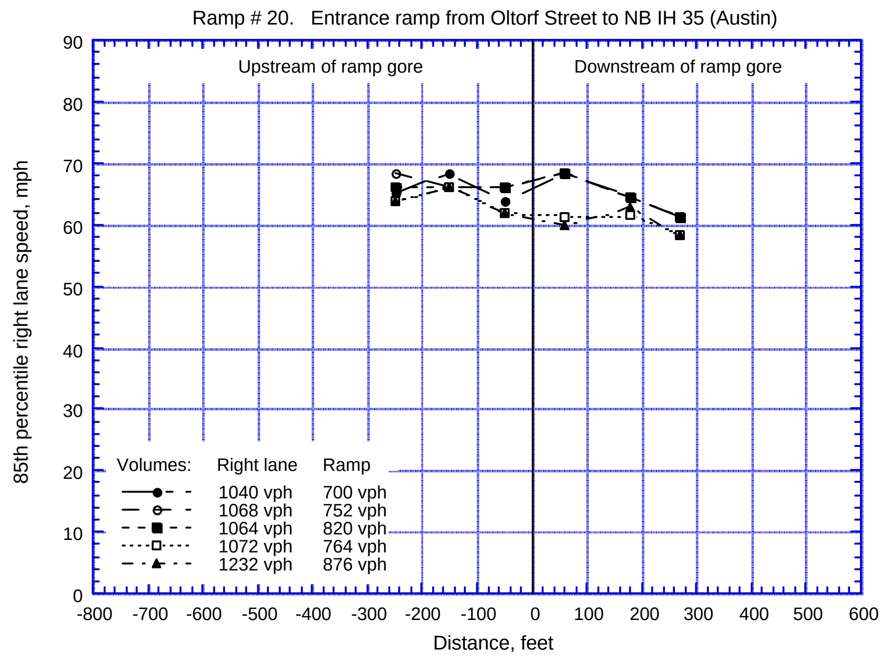

Figure F03 85th Percentile Freeway Right Lane Speed, Ramp #20
Entrance Ramp from Oltorf Street to NB IH 35, Austin

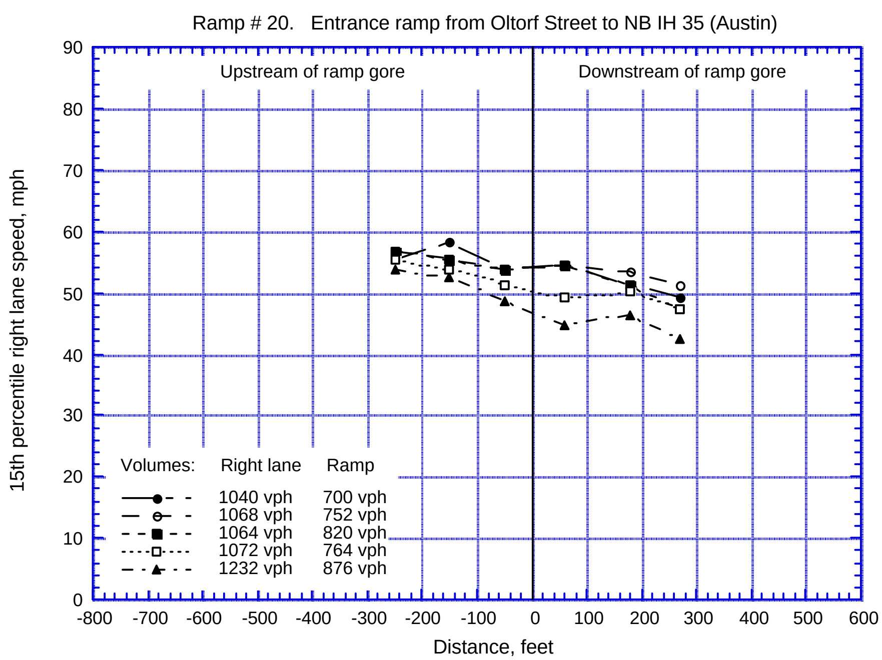

Figure F04 15th Percentile Freeway Right Lane Speed, Ramp #20
Entrance Ramp from Oltorf Street to NB IH 35, Austin

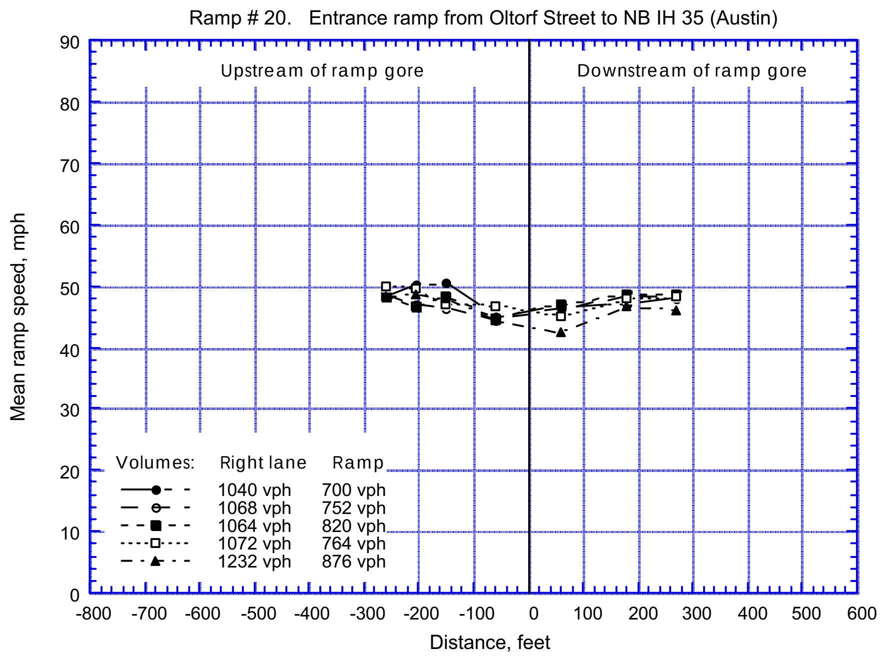

Figure F05 Mean Ramp Speed, Ramp #20 Entrance
Ramp from Oltorf Street to NB IH 35, Austin

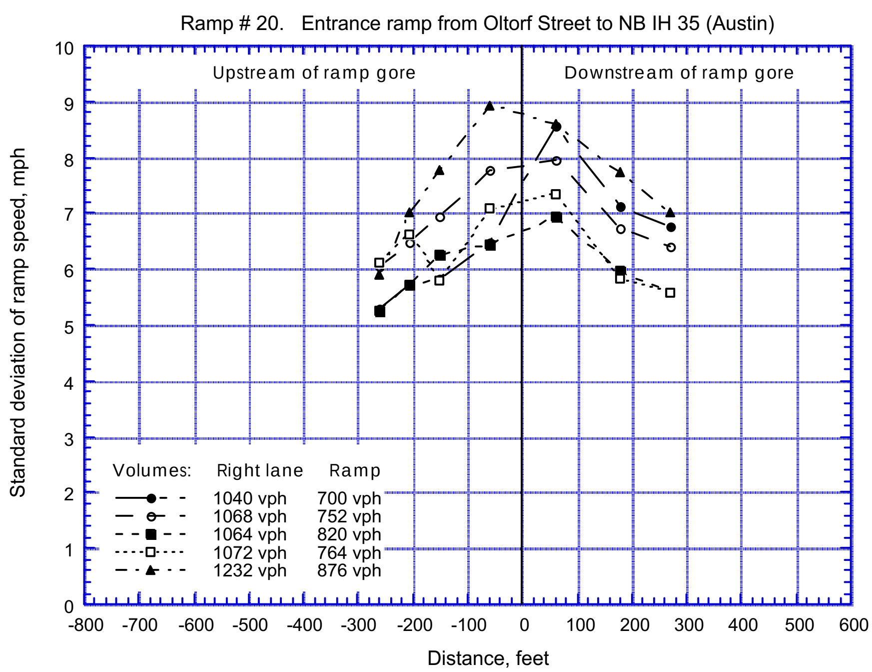

Figure F06 Standard Deviation of Ramp Speed, Ramp #20
Entrance Ramp from Oltorf Street to NB IH 35, Austin

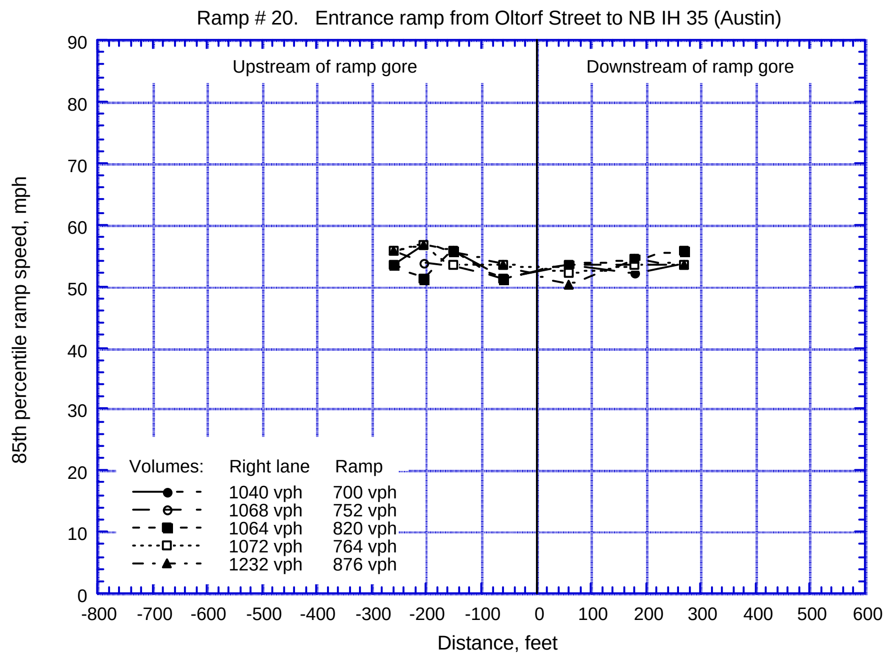

Figure F07 85th Percentile Ramp Speed, Ramp #20 Entrance Ramp from Oltorf Street to NB IH 35, Austin

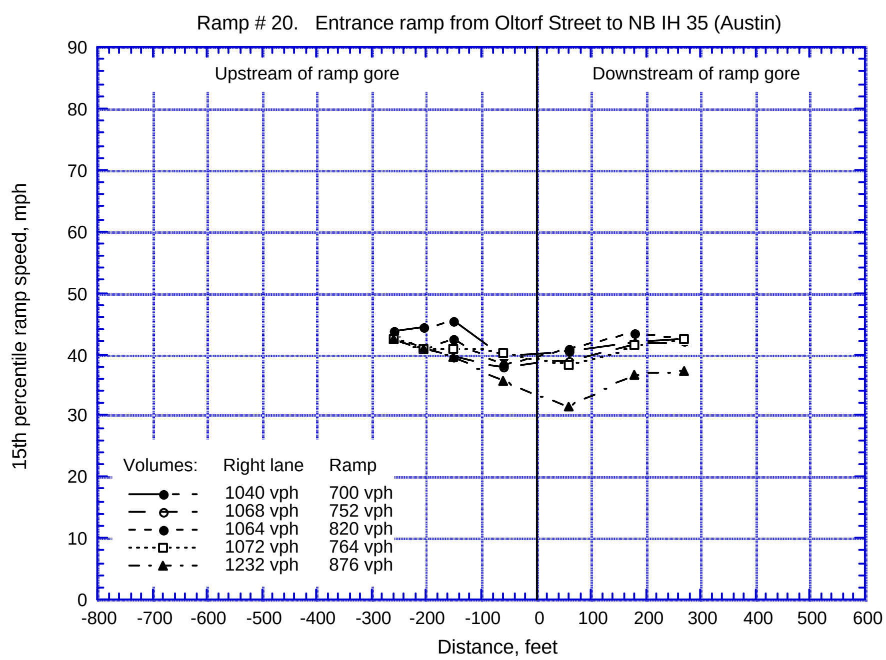

Figure F08 15th Percentile Ramp Speed, Ramp #20 Entrance Ramp from Oltorf Street to NB IH 35, Austin

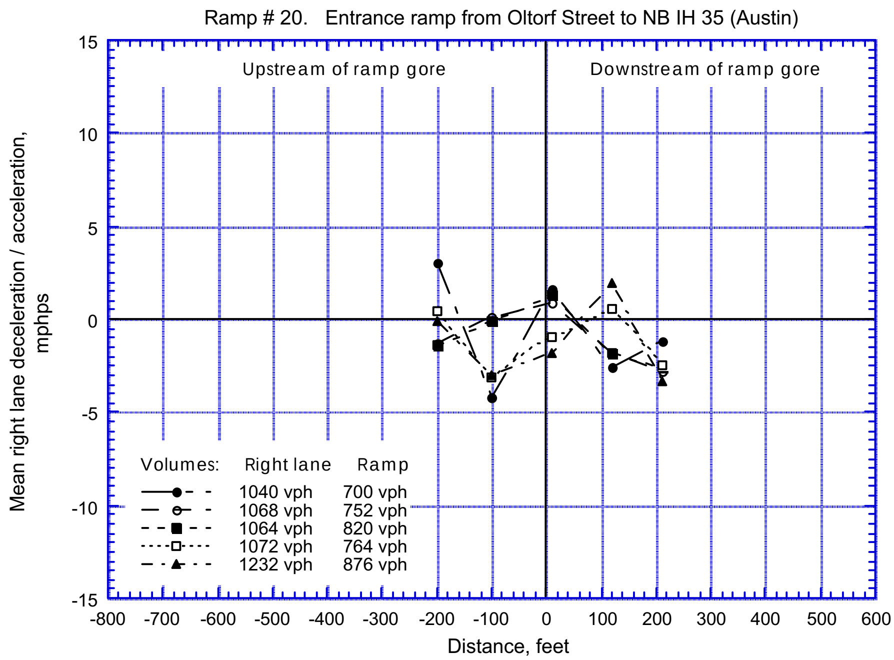

Figure F09 Mean Freeway Right Lane Acceleration/Deceleration,
Ramp #20 Entrance Ramp from Oltorf Street to NB IH 35, Austin

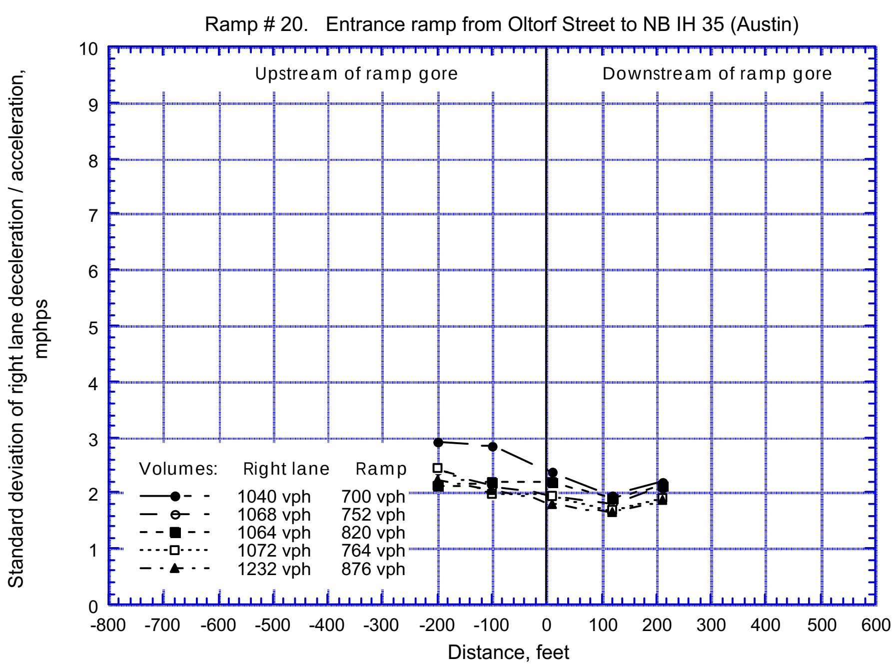

Figure F10 Standard Deviation of Freeway Right Lane Acceleration/Deceleration,
Ramp #20 Entrance Ramp from Oltorf Street to NB IH 35, Austin

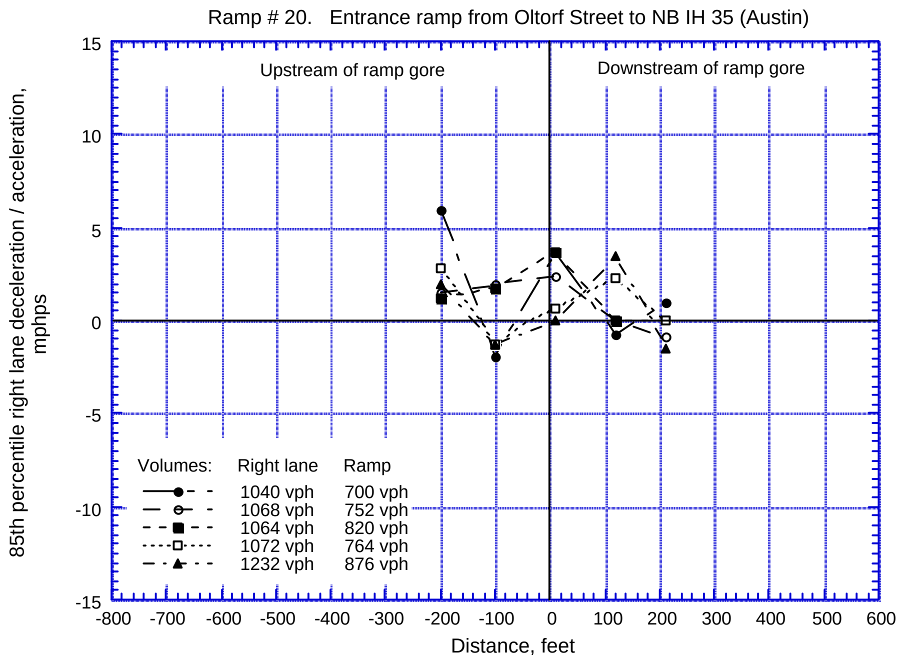

Figure F11 85th Percentile Freeway Right Lane Acceleration/Deceleration, Ramp #20 Entrance Ramp from Oltorf Street to NB IH 35, Austin

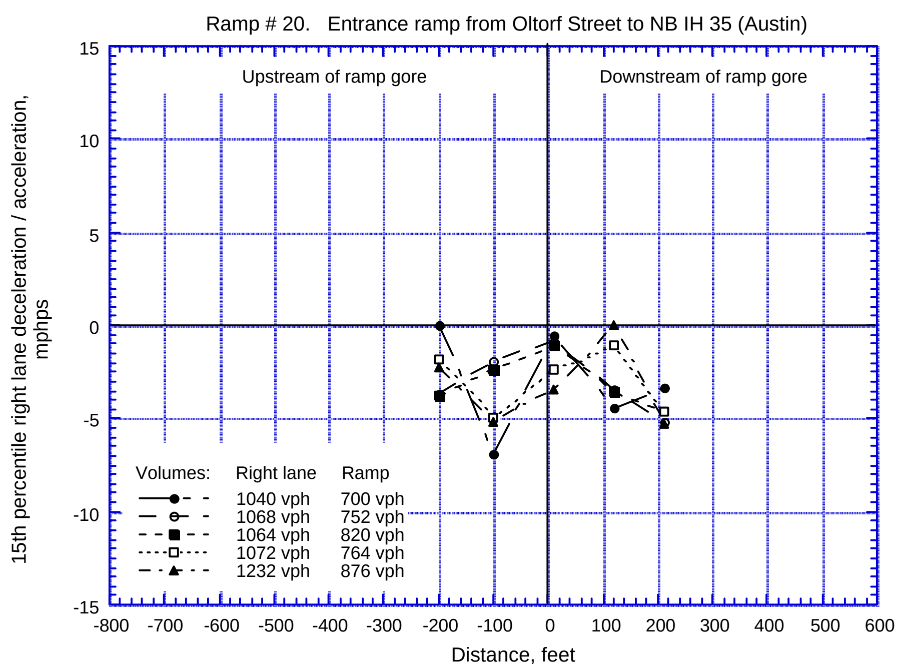

Figure F12 15th Percentile Freeway Right Lane Acceleration/Deceleration, Ramp #20 Entrance Ramp from Oltorf Street to NB IH 35, Austin

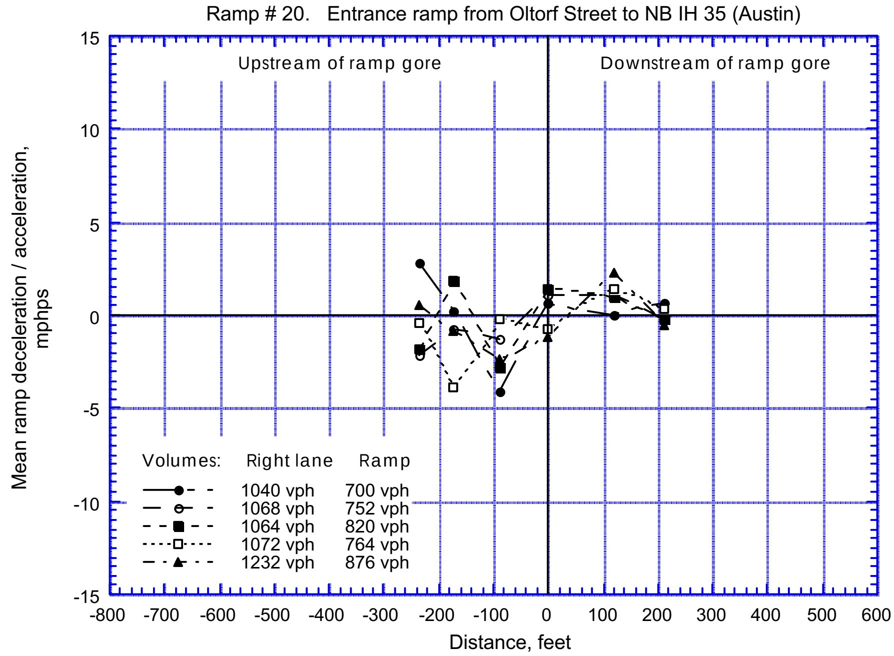

Figure F13 Mean Ramp Acceleration/Deceleration, Ramp #20
Entrance Ramp from Oltorf Street to NB IH 35, Austin

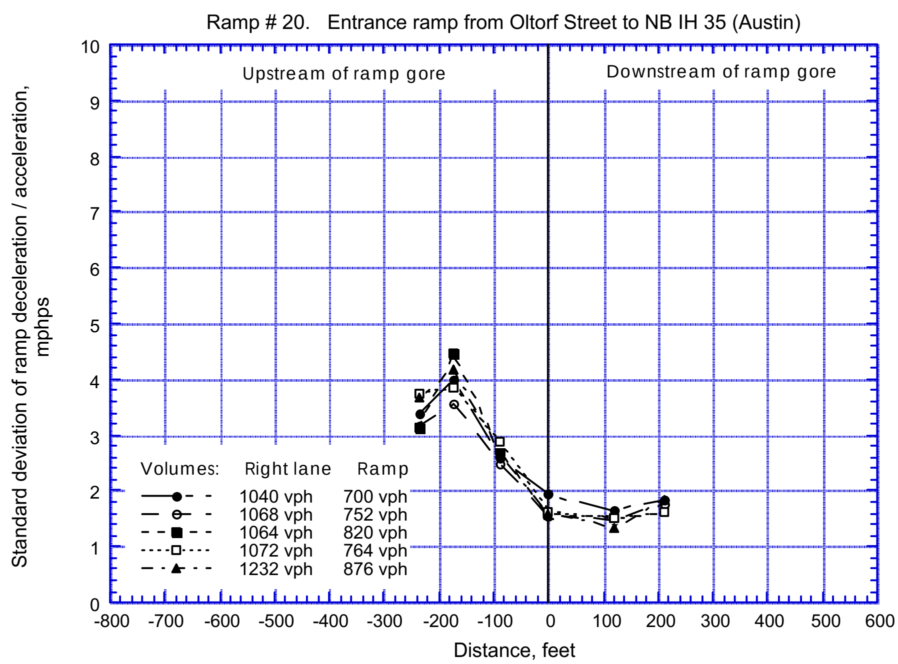

Figure F14 Standard Deviation of Ramp Acceleration/Deceleration,
Ramp #20 Entrance Ramp from Oltorf Street to NB IH 35, Austin

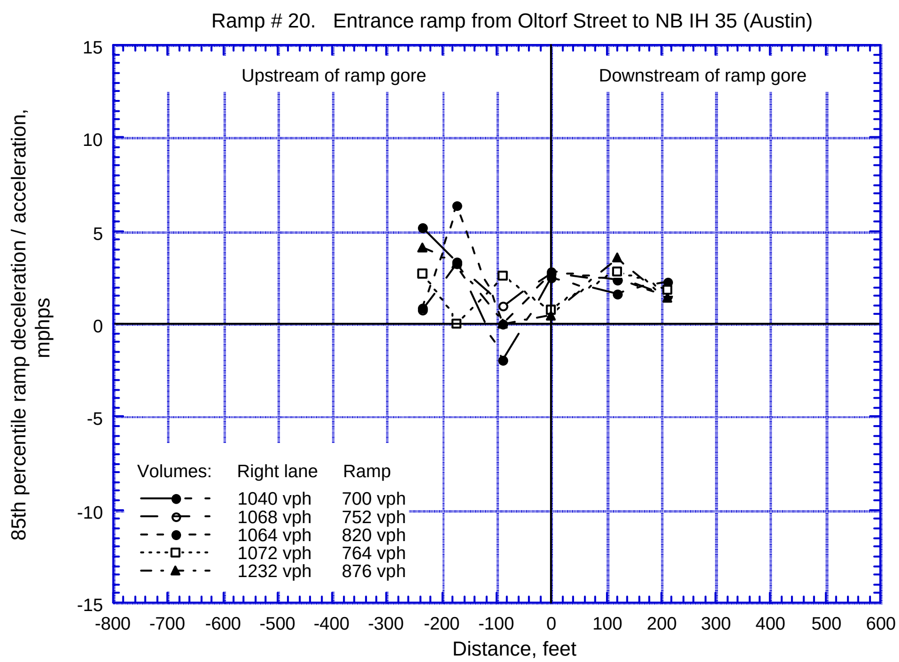

Figure F15 85th Percentile Ramp Acceleration/Deceleration, Ramp #20
Entrance Ramp from Oltorf Street to NB IH 35, Austin

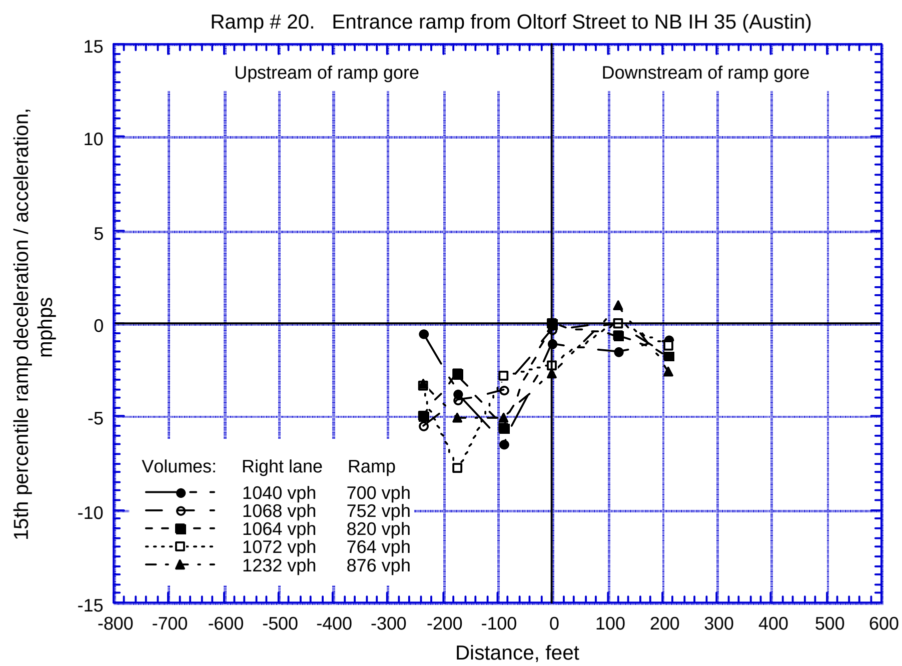

Figure F16 15th Percentile Ramp Acceleration/Deceleration, Ramp #20
Entrance Ramp from Oltorf Street to NB IH 35, Austin

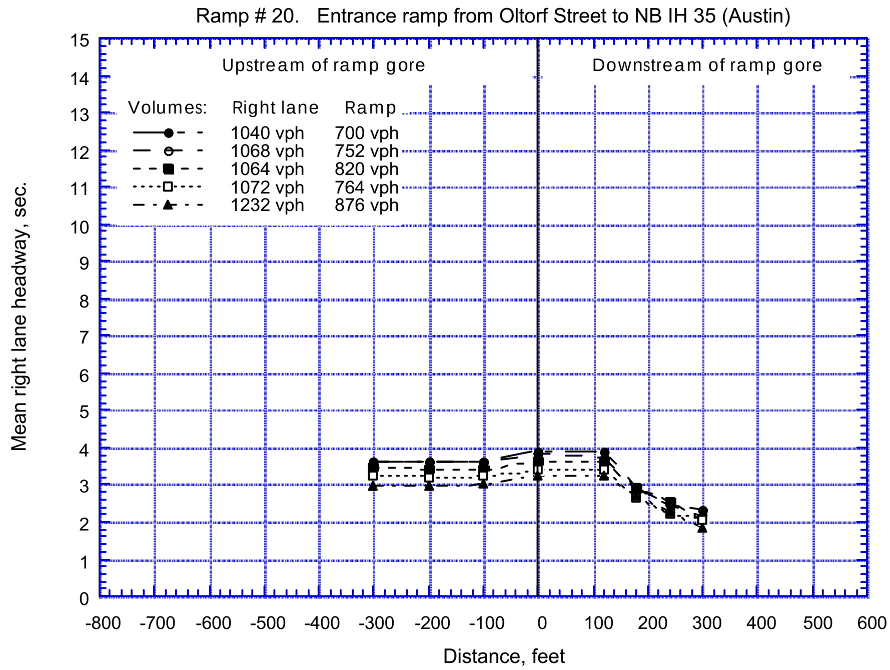

Figure F17 Mean Time Headway Freeway Right Lane, Ramp #20
Entrance Ramp from Oltorf Street to NB IH 35, Austin

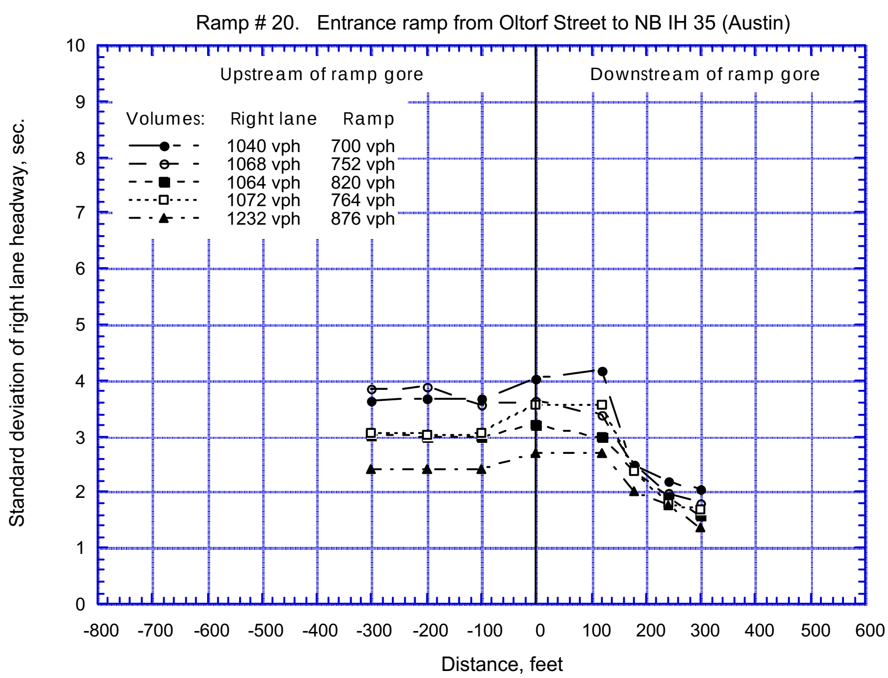

Figure F18 Standard Deviation of Time Headway Freeway Right Lane,
Ramp #20 Entrance Ramp from Oltorf Street to NB IH 35, Austin

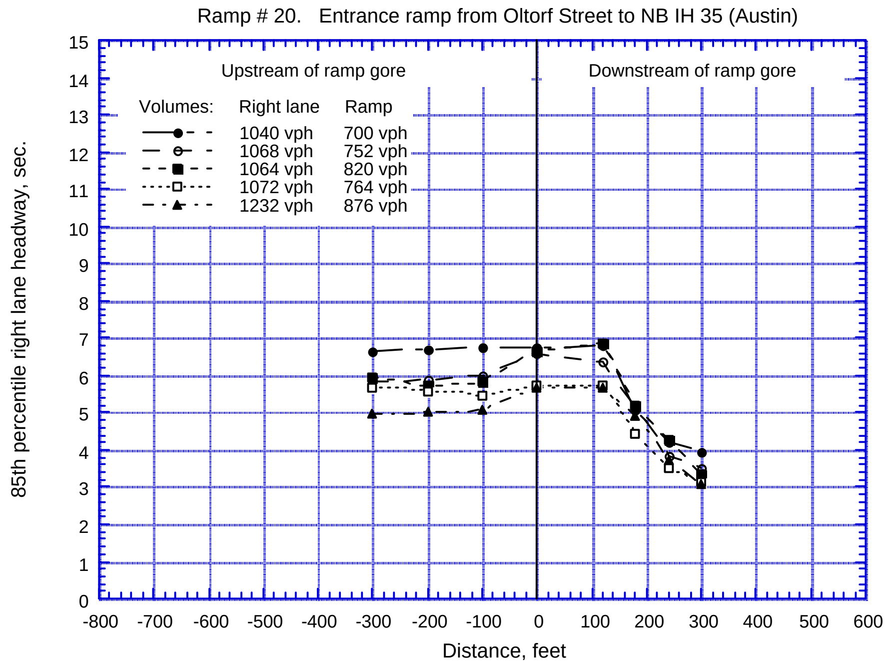

Figure F19 85th Percentile Time Headway Freeway Right Lane, Ramp #20
Entrance Ramp from Oltorf Street to NB IH 35, Austin

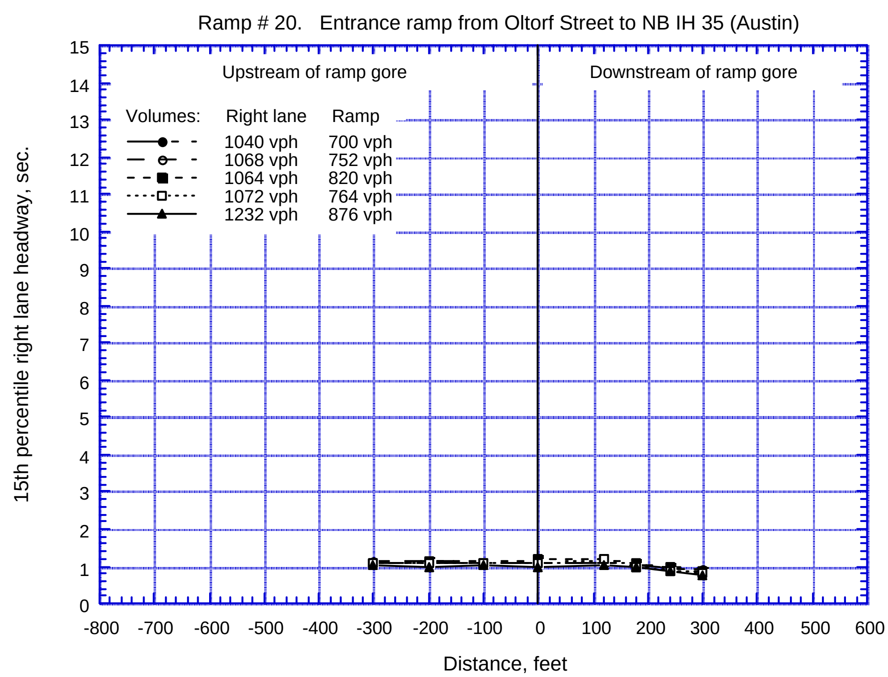

Figure F20 15th Percentile Time Headway Freeway Right Lane, Ramp #20
Entrance Ramp from Oltorf Street to NB IH 35, Austin

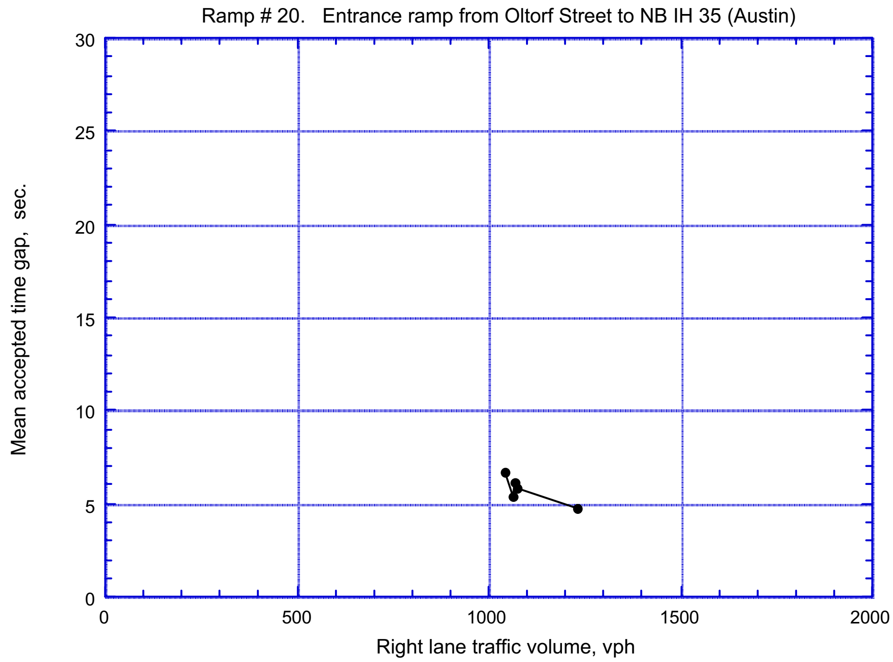

Figure F21 Mean Freeway Time Gap Accepted by Ramp Vehicles Versus Right Freeway Lane Traffic Volume, Ramp #20 Entrance Ramp from Oltorf Street to NB IH 35, Austin

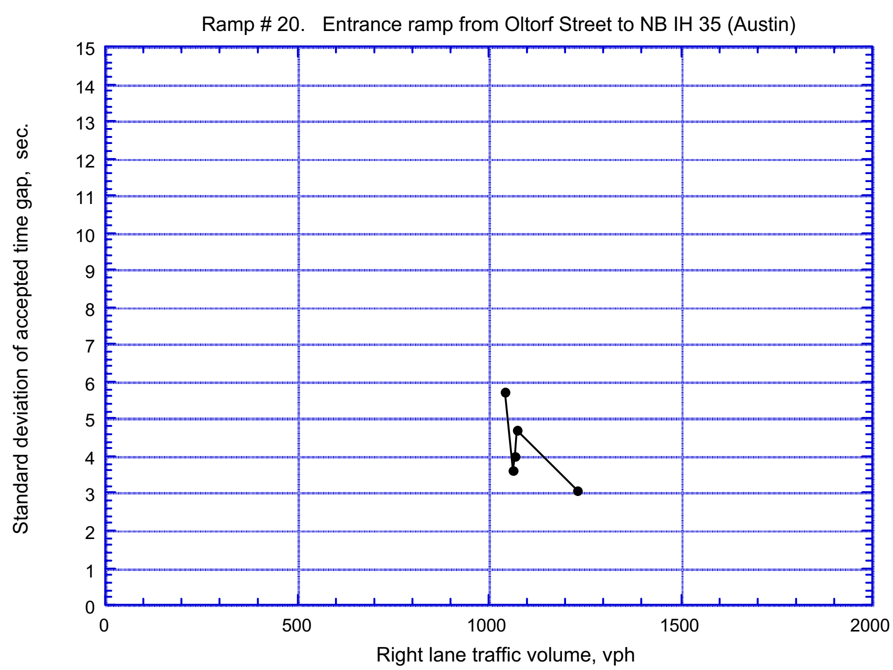

Figure F22 Standard Deviation of Freeway Time Gap Accepted by Ramp Vehicles, Ramp #20 Entrance Ramp from Oltorf Street to NB IH 35, Austin

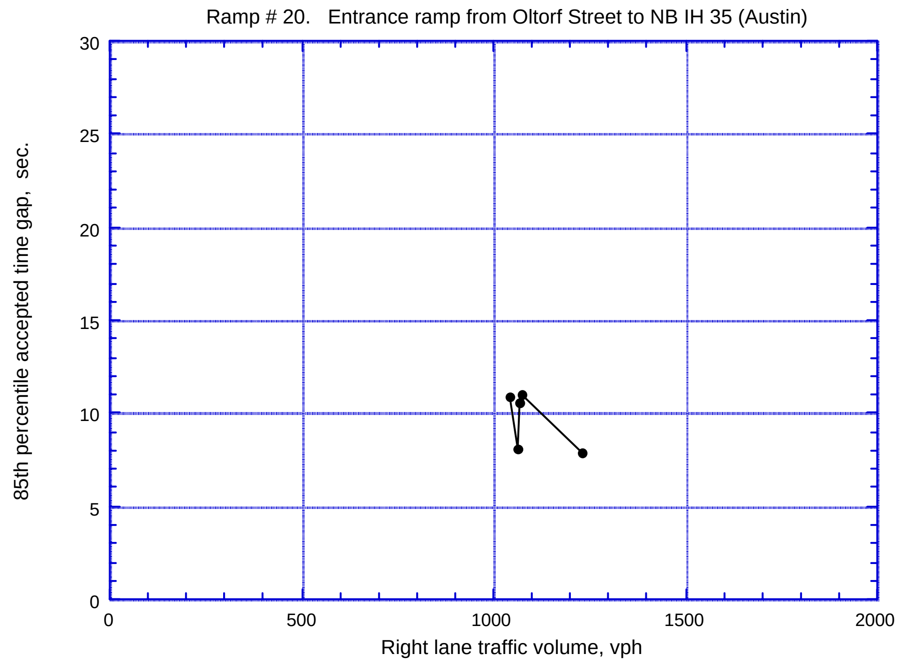

Figure F23 85th Percentile Freeway Time Gap Accepted by Ramp Vehicles,
Ramp #20 Entrance Ramp from Oltorf Street to NB IH 35, Austin

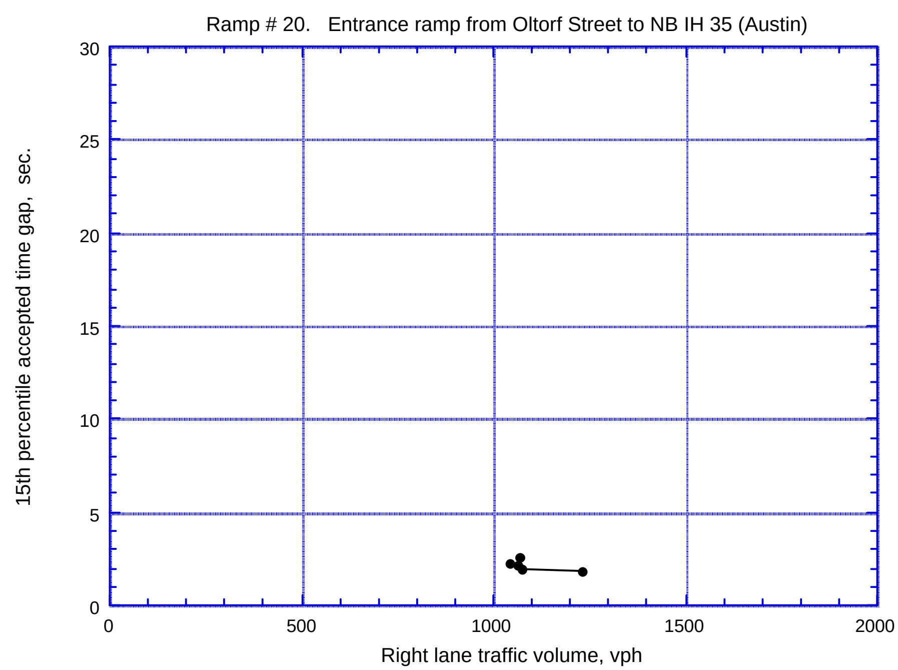

Figure F24 15th Percentile Freeway Time Gap Accepted by Ramp Vehicles,
Ramp #20 Entrance Ramp from Oltorf Street to NB IH 35, Austin

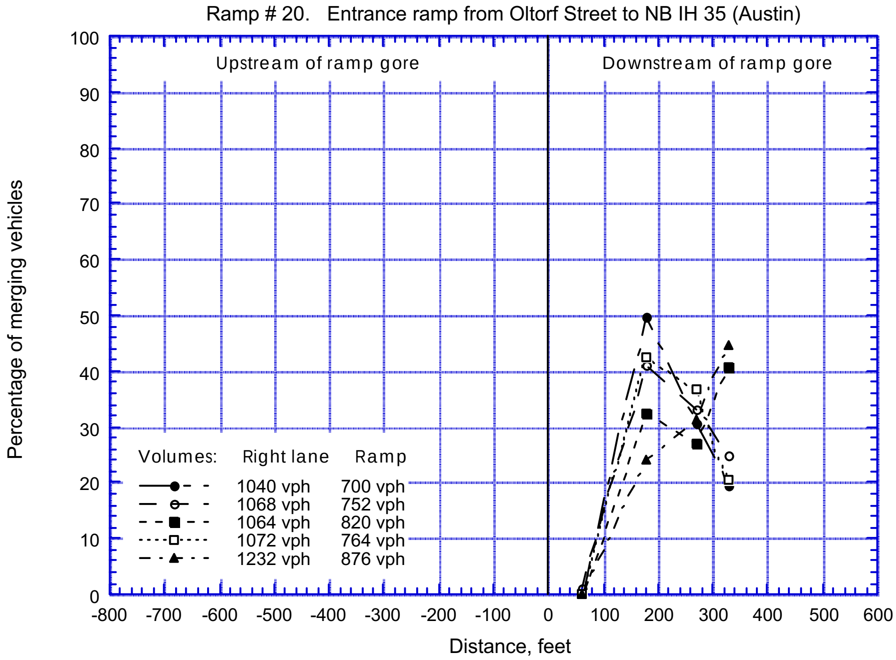

Figure F25 Ramp Vehicle Merging Location Percentage, Ramp #20
Entrance Ramp from Oltorf Street to NB IH 35, Austin

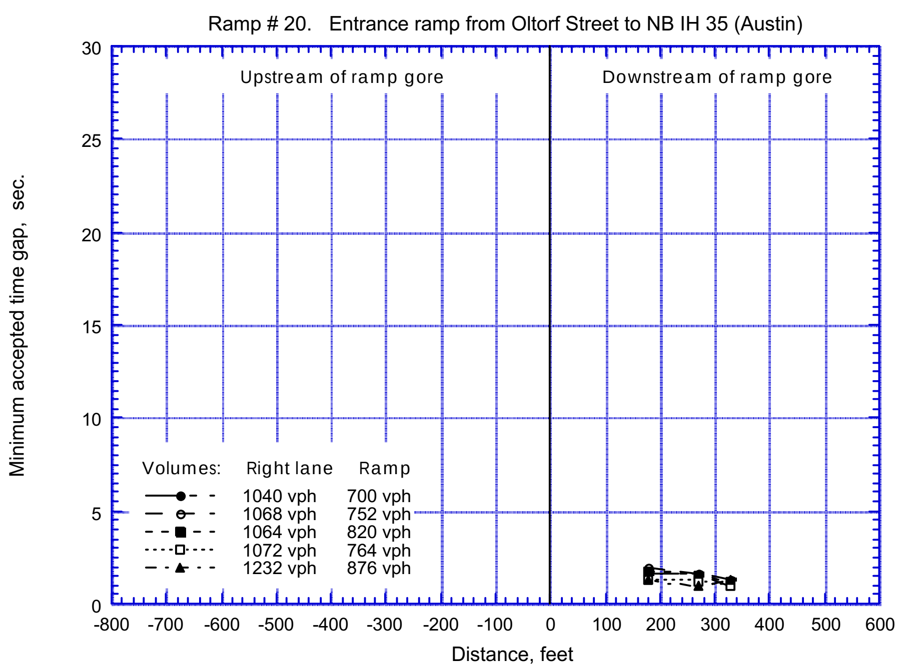

Figure F26 Minimum Time Gap Accepted by Ramp Vehicles, Ramp #20
Entrance Ramp from Oltorf Street to NB IH 35, Austin

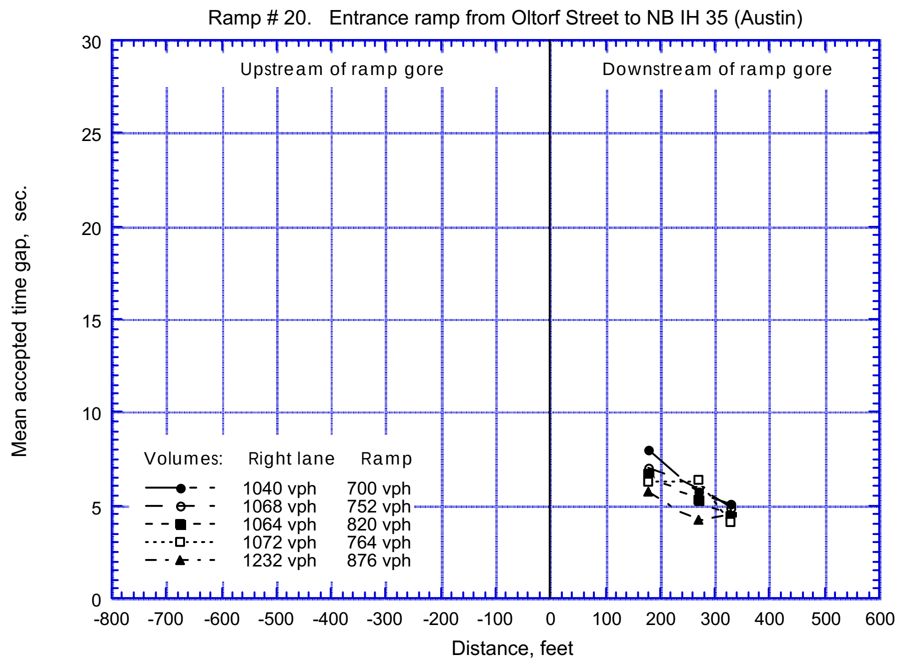

Figure F27 Mean Freeway Time Gap Accepted by Ramp Vehicles Versus Merging Location, Ramp #20 Entrance Ramp from Oltorf Street to NB IH 35, Austin

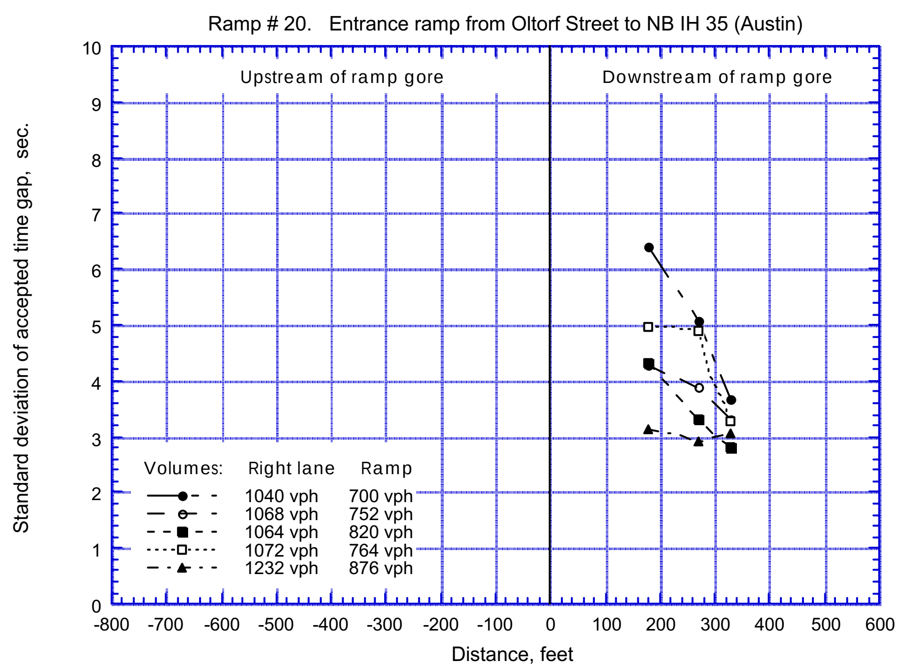

Figure F28 Standard Deviation of Freeway Time Gap Accepted by Ramp Vehicles Versus Merging Location, Ramp #20 Entrance Ramp from Oltorf Street to NB IH 35, Austin

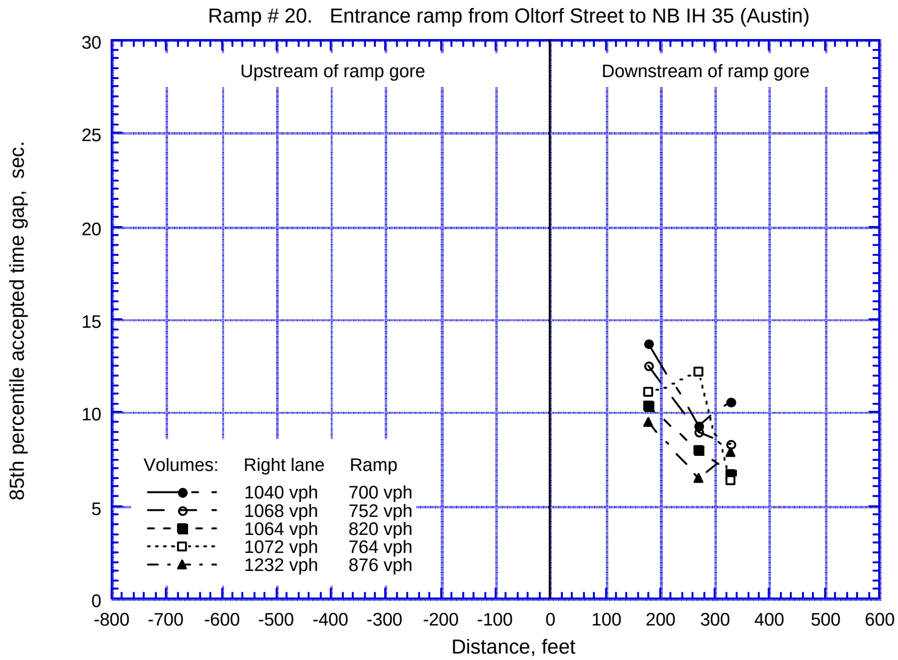

Figure F29 85th Percentile Freeway Time Gap Accepted by Ramp Vehicles Versus Merging Location, Ramp #20 Entrance Ramp from Oltorf Street to NB IH 35, Austin

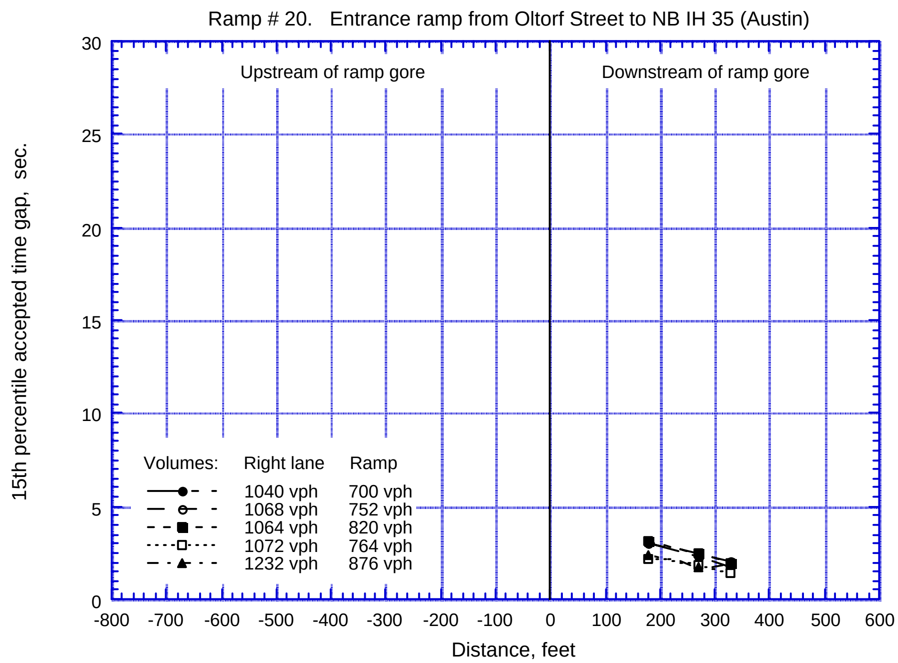

Figure F30 15th Percentile Freeway Time Gap Accepted by Ramp Vehicles Versus Merging Location, Ramp #20 Entrance Ramp from Oltorf Street to NB IH 35, Austin